COSTS.

Fig. 192 gives the cost per foot of reinforced concrete penstock. The data from which this curve is plotted was derived largely from Gillette's book, " Cost Data," but also from numerous other sources.

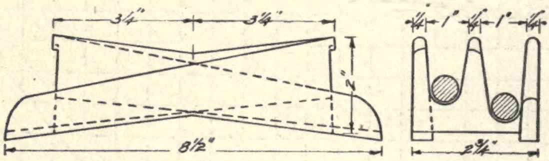


Fig. 191.

Reinforcing costs about three cents per pound for steel, and 0.5 cent to instal. The concrete costs bout $10.00 per cubic yard, including every item. Round rods cost about

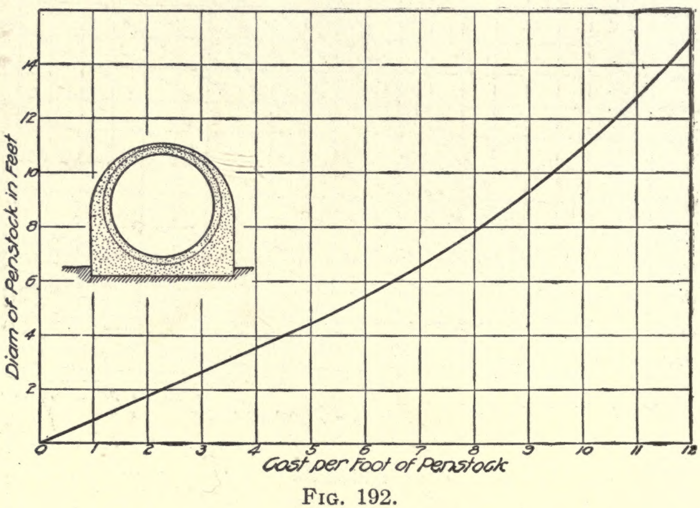


Fig. 192.

$34.00 per ton. Brick penstocks require 570 bricks per cubic yard and 1.25 barrels of cement. A mason should lay 1200 bricks in eight hours at a cost of $6.00

Figs. 193 to 195 show the three common forms of steel riveted

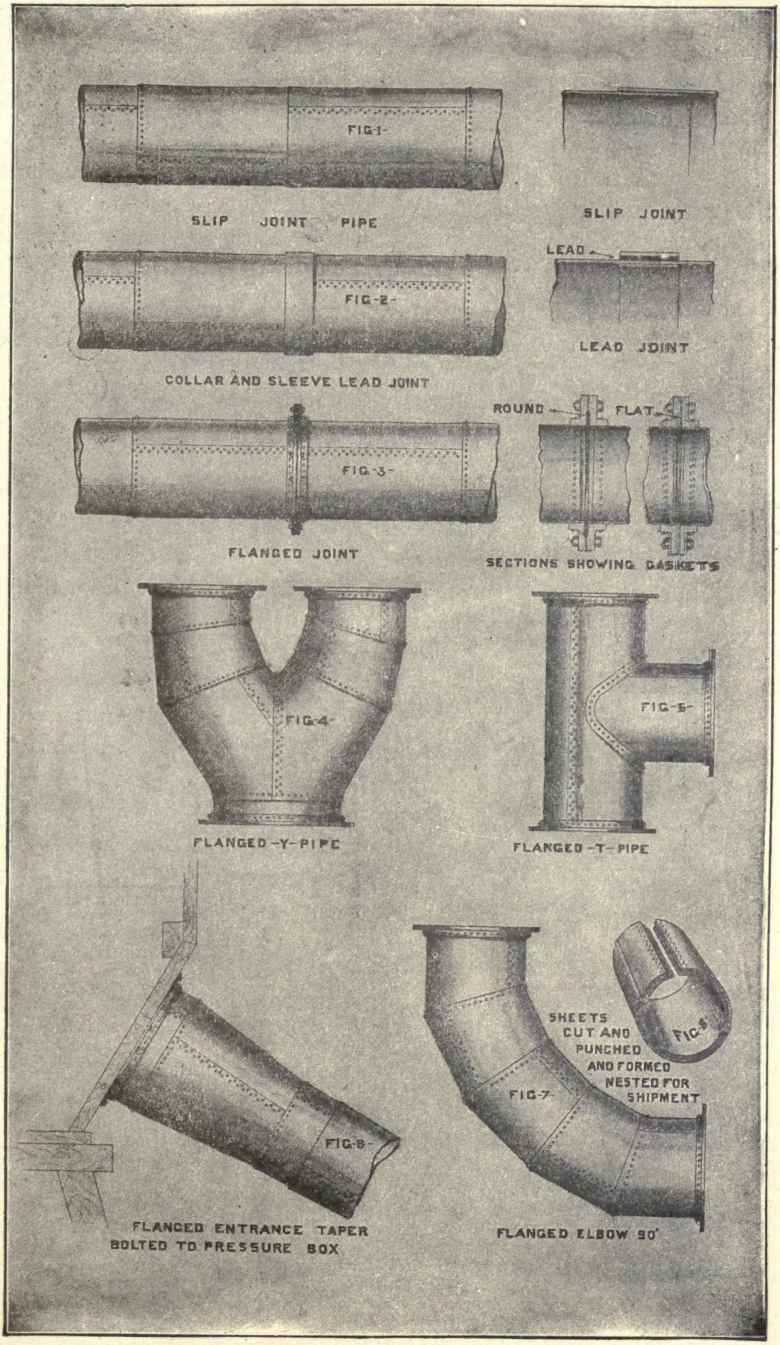


FIG-1-
SLIP JOINT PIPE
SLIP JOINT
LEAD
FIG-2-
LEAD JOINT
COLLAR AND SLEEVE LEAD JOINT
ROUND
FLAT
FIG-3-
FLANGED JOINT
SECTIONS SHOWING GASKETS
FIG-4-
FIG-5-
FLANGED -Y- PIPE
FLANGED -T- PIPE
SHEETS
CUT AND
PUNCHED
AND FORMED
NESTED FOR
SHIPMENT
FIG-6-
FIG-7-
FIG-8-
FLANGED ENTRANCE TAPER
BOLTED TO PRESSURE BOX
FLANGED ELBOW 90°

FIGS. 193–195.


UNIVERSITY OF CALIFORNIA
ANDREW SMITH HALLIDIE:
REGENT
1868 1901

DESIGN AND CONSTRUCTION
OF
HYDROELECTRIC PLANTS

INCLUDING A SPECIAL TREATMENT
OF THE
DESIGN OF DAMS

BY

R. C. BEARDSLEY

NEW YORK
McGRAW PUBLISHING COMPANY
1907

To my father, E. R. Beardsley,
to whom we owe the gravity dam,
and the discovery of the existence
and effects of vacuums under dams.

CONTENTS.

CHAPTER I

HYDRAULIC PRINCIPLES.

Hydrostatics.

Water is chemically known as H_2O. Its weight varies from 62.3 pounds to 62.5 pounds per cubic foot.

In all estimates on water power the value should be used which gives results on the safe side. Thus, in finding the power of the stream 62.3 should be used, but in obtaining the pressure on the dam or against a gate, 62.5 is the safest figure. In turbine testing, where great accuracy is required, the water should be weighed during the test.

1 cubic foot of water = 6.232 imperial gallons = 7.48 United States gallons.

1 imperial gallon = .1605 cubic foot = 1.2 United States gallons.

1 United States gallon = .1337 cubic foot = .331 imperial gallons.

1 United States gallon = 8.355 pounds = 231 cubic inches.

1 miners inch = 11.219 United States gallons = 1.5 cubic foot.

The miner's inch is not the same in all parts of the country, but the value given above is becoming universally acknowledged.

Water is but slightly compressible, therefore the pressure, P, is for all practical purposes directly proportional to the depth, H, and can be represented by a diagram as shown in Figs. 1, 2 and 3.

The *total pressure* on any submerged surface is equal to the area of the pressure diagram ($a\ b\ c$, Fig. 1; $d\ e\ c\ b$, Figs. 2 and 3), and the center of pressure passes through its center of gravity G perpendicular to the submerged surface. The *moment* of the pressure about c is

$$M = P\,y.$$

The *pressure* in pounds per square inch at any point is

$$p = 0.433 \, Depth.$$

The *pressure is always normal* to the submerged surface (Fig. 4). The total pressure exerted on a submerged body is

$$P = 0.433 \, H \, S$$

wherein S is the area of the surface and H the depth of water over the geometrical center of the body.

Any *body submerged* in water will suffer an apparent loss of weight which is equal to the weight of the displaced volume of water. If a unit volume of water is heavier than a unit volume of the substance the latter will float.

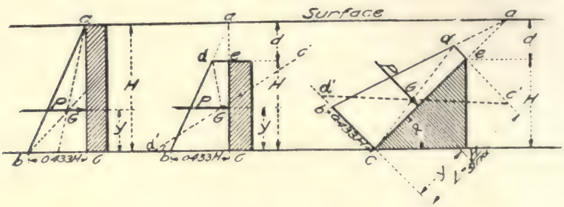


FIG. 1. FIG. 2. FIG. 3.

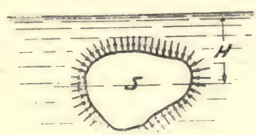


FIG. 4.

HYDRODYNAMICS.

Water in motion is governed by the same law as falling bodies, *i.e.*,

$$m \, g \, h = \frac{m \, v^2}{2}$$

wherein $m \, g \, h$ represents the potential energy due to its position and $\frac{m \, v^2}{2}$ represents the kinetic energy due to its velocity. This equation holds true only for an efficiency of 100 per cent.

The quantities which enter into the equation are the mass, m, head, h, velocity, v, and the gravity constant, $g = 32.16$.

$$m\,g = w = \text{weight}$$

and

$$h = \frac{v^2}{2g}$$

$$v = \sqrt{2\,g\,h} = 8.03\sqrt{h}$$

When h is given in feet, v is the velocity in feet per second.

The *flow of water through an opening* expressed in cubic feet per second is

$$Q = v\,A$$

wherein v is the velocity in feet per second and A is the area in square feet, of the opening.

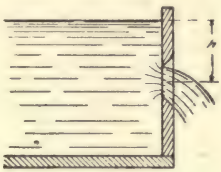


FIG. 5.

For *rectangular openings* of length, l, and depth, d, the formula for Q is

$$Q = c\,l\,d\sqrt{2\,g\,h} \quad \text{(Fig. 5)}$$

For *circular openings* of diameter d the formula for Q is (Fig. 5)

$$Q = c\frac{\pi\,d^2}{4}\sqrt{2\,g\,h}\left(1 - \frac{1}{128}\frac{d^2}{h^2} - \frac{5}{16384}\frac{d^4}{h^4} - \ldots\right)$$

for values of $h > 2\,d$

$$Q = c\frac{\pi\,d^2}{4}\sqrt{2\,g\,h}$$

wherein c is a *coefficient* which depends on the form of the orifice and may be taken as 0.61 for openings in thin plates or planks such as head gates; Q is given in cubic feet per second when h, l and d are measured in feet.

The above formulas for flow through orifices supposed that there is no velocity of approach and are correct to within 0.5 per cent. when

$$\frac{A'}{A} > 10$$

wherein A′ is the area of cross-section of the canal or tank and A that of the orifice.

When

$$\frac{A'}{A} < 10,$$

The *velocity of approach* should be taken into consideration. Let h_0 be the head due to the velocity of approach, *i.e.*,

$$h_0 = \frac{v^2}{2g}$$

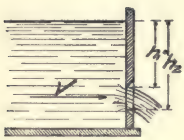


FIG. 6.

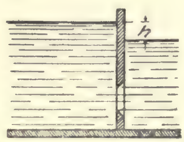


FIG. 7.

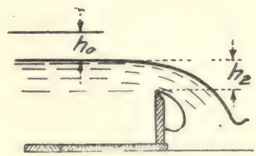


FIG. 8.

Then

$$Q = c\,\tfrac{2}{3}\,l\sqrt{2g}\,[(h_2+h_0)^{\frac{3}{2}} - (h_1+h_0)^{\frac{3}{2}}] \qquad \text{(Fig. 6.)}$$

$$c = 0.61$$

for rectangular openings of length l.

For *submerged orifices* the discharge is practically the same as in the case of a free discharge except that the head, h, is taken between the two levels (Fig. 7).

$$Q = c\,A\sqrt{2\,h\,g}$$
$$c = 0.6$$

The *weir* is a special case of a rectangular orifice where $h_1 = 0$.

$$Q = \tfrac{2}{3}\,c\,l\sqrt{2g}\,(h_2 + n\,h_0)^{\frac{3}{2}} \qquad \text{(Fig. 8.)}$$

$$n = 1.0 - 1.5 \qquad c = 0.60$$

Suppose water to be conducted through a pipe line from

one reservoir to another. The difference between the levels being h (Fig. 9). If pressure tubes are inserted at intervals along the pipe line their levels will coincide with the line $a\ b$ called the *hydraulic gradient* when the line is open and with the line $a\ a'$ when the valve at B is closed. This reduction in pressure head is due to two things, namely, friction losses and conversion of pressure head into velocity head. The velocity head is represented by h_v and

$$h_v = \frac{v^2}{2\,g}$$

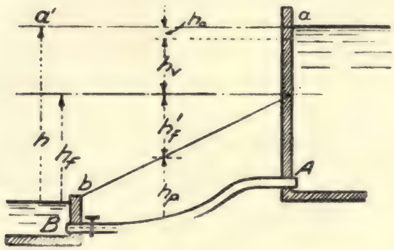


FIG. 9.

The *friction head* lost at the entrance to the pipe is expressed thus

$$h_0 = c\,\frac{v^2}{2\,g}$$

$$c = 0.5 \text{ (approximately)}$$

The *friction head* lost in the pipe is directly proportional to the length of the pipe, inversely proportional to the diameter of the pipe, directly proportional to the square of the velocity and is expressed thus

$$h_f = c\,\frac{l}{d}\,\frac{v^2}{2\,g}$$

c is the coefficient of friction.

The total available static head is

$$h = h_0 + h_f + h_v$$

From Figs. 9 and 10 it is seen that a pipe laid along the hydraulic gradient would not be subjected to pressure except when the pipe is closed at the lower end and open at the upper,

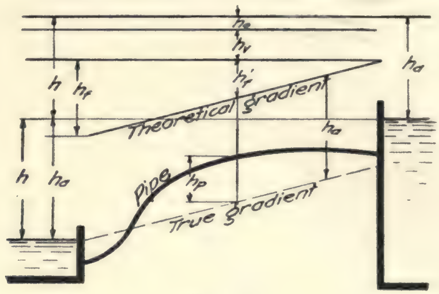


FIG. 10.

and that at all portions of the pipe line which lie below the hydraulic gradient are subjected to the pressure h_p from the *inside* while those that lie above it are subjected to the pressure h_p from the *outside*.

When the pipe line rises above the hydraulic gradient it is

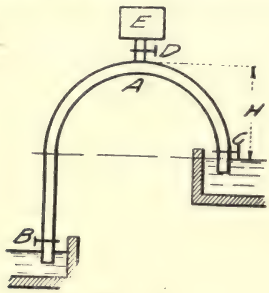


FIG. 11.

called a *siphon*. A siphon requires an air tight pipe because its operation depends upon the possibility of raising the hydraulic gradient by an amount equal to the head due to the atmosphere as shown in Fig. 10. Even though the pipe be air tight some air will be carried in by the water and will collect in the pipe at the highest point. At this point a tank with

a valve must be inserted to collect and carry off the air. It is also used to start the siphon. The operation is explained as follows (Fig. 11):

To operate the pipe, the valves at *B* and *C* are closed, *D* opened and the whole siphon and reservoir *E* filled with water. *B* and *C* are now opened, *D* being left open. Then, as the air forms during the operation of the pipe, the air drives the water out of *E*, but *E* having some capacity, it requires time to do this and, before *E* is entirely emptied, the valve *D* is closed and *E* filled with water, after which *D* is opened again. In this way, the siphon may be caused to operate continuously.

If the head lost by friction in the pipe exceeds 20 to 25 feet the pipe will not operate successfully.

In a siphon, the pressure to be contended with is that due to air pressure and the pipe must be strong enough against collapsing to stand 15 pounds per square inch.

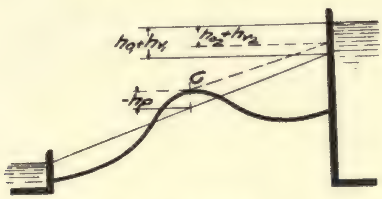


FIG. 12.

Siphons may form a part of the penstock when it is cheaper than tunnelling. Large siphons may be built of wooden staves though riveted steel pipe is, in all cases, preferable.

From Fig. 10 it is seen that the hydraulic gradient is raised by an amount equal to the head due to the atmospheric pressure and that the pipe should work as well above as below the apparent hydraulic gradient. This is true enough in theory, but in practice pipes will leak and allow air to enter and the water will carry air which will collect at the high points and form air plugs, and shift the hydraulic gradient from its normal position to that shown in Fig. 12, so that the part of the pipe above the point *C* will be under pressure and the discharge will take place at *C*, the rest of the pipe acting simply as a channel to convey the water from there to the end. This will cause a material decrease in the velocity and consequently the flow of the water. Water

carried from a great height is often utilized at a nozzle by absorbing its kinetic energy as in a Pelton wheel. In this case the velocity is kept low in the pipe so as to reduce the head lost in friction and the greater part of velocity head developed in a nozzle. Fig. 13 shows the hydraulic gradient for a case of this sort. In the cases which went before, the gradient was taken as a straight line from one end of the pipe line to the other, but this is only true where the pipe itself follows approximately a straight line between the reservoir and the discharge.

The *impulse pressure* at the nozzle is obtained by assuming that this velocity was produced by the action of a force F for a period of one second; then the product of the force, F, and the distance, $\frac{v}{2}$, through which the weight W moves in one

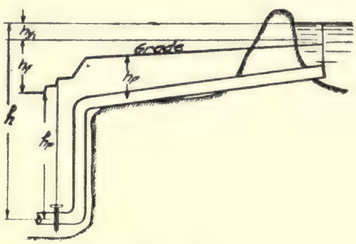


FIG. 13.

second, is the work and is equal to the kinetic energy, thus

$$F\frac{v}{2} = W\frac{v^2}{2g}$$

$$F = W\frac{v}{g} = wq\frac{v}{g} = wa\frac{v^2}{g} = 2wa\frac{v^2}{2g}$$

wherein q is the volume per second, w the specific weight of water, and a the area of the orifice. Thus, it is seen that were there no losses the head, h, corresponding to the velocity, v, would produce an impulse pressure equal to that produced by a static head, $2h$. This demonstrates that the sudden closing of a valve in a pipe line may subject the pipe to enormous pressure.

The *time*, T, which it takes the water to attain its final velocity in a frictionless pipe or to come to rest when a valve is closed is,

$$T = 0.249 \sqrt{l} \text{ seconds,}$$

wherein l is the length of the pipe in feet.

The *kinetic energy* in foot pounds possessed by a pipe full of water in motion is,

$$\begin{aligned} E_R &= 0.765\, d\, l\, v^2 = h \\ &= 49.2\, d\, l\, h \end{aligned}$$

wherein d is the diameter of the pipe in feet, l the length in feet, h the head in feet, and v the velocity in feet per second.

CHAPTER II.

MEASUREMENT OF FLOW

The flow is the amount of water which passes a given point in a given time, and is determined by substituting experimental constants in theoretical formulas which are derived for the aperture through which it is desired to measure the flow.

WEIRS.

Weirs are used in measuring the flow in small streams or the discharge of turbines, pumps, etc.

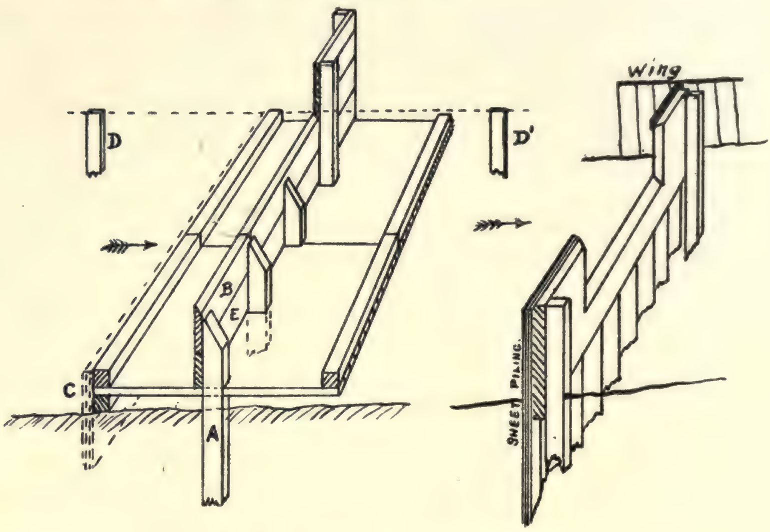


FIG. 14.—Wire for small streams.

The author has found that the construction shown in Fig. 14 is very satisfactory when the flow of small streams is to be measured. The weir may be of any length or size, and built on any bottom and is easily and quickly constructed in swift water.

The posts, A, are driven with a maul at equal distances apart across the stream and as nearly as possible in line.

Then the floor is made on shore, in sections of, say, 12 feet in length and as wide as thought necessary, for that particular bottom and height of weir. Holes are cut in the floor so that it may be dropped down over the posts.

The sections are then placed over the posts and sunk on to the bed of the stream and weighted down with rock. The tops of the posts are cut tapering and all on the same level except the four end posts which are left long to form the abutments. Then the first plank, E, is fitted to the floor so that its upper edge is perfectly level. If the river bed is of sand a row of sheet piling must be driven at C before the floor or mat is placed, but for most bottoms this will not be necessary. Having placed the plank, E, fill above it with earth to prevent the water cutting under the floor when the head is increased.

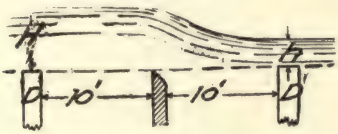


Fig. 15.

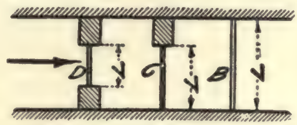


Fig. 15a.

The height of these planks will depend on the depth of the water as the top or weir plank must be at least a foot above the lower water level. Champfer the edge of the weir plank, B, to a thin edge, and place plenty of earth at the ends. A stake, D, is driven up stream a distance of six feet or more from the we r so that its top is exactly on a level with the top edge of the weir plank.

The total cubic feet of water per minute flowing in the stream is obtained by measuring the depth of water over the stake, D, and from the weir table (see Table II) finding the cubic feet of water flowing each minute per foot width of weir then multiplying this quantity by the width of the weir in feet. Since a cubic foot of water weighs 62.5 pounds, the pounds of water flowing per minute are equal to the flow in cubic feet per minute, times 62.5.

A formula which takes end contraction into account is as follows:

Q, the cubic feet of water flowing over the weir per minute = 199.8 $(L - 0.1\, n\, D)\, D\, 3/2$.

D = depth of water in feet above A, measured at a point some six feet to ten feet up stream from the weir. n = the number of end contractions. Thus in Fig. 15a at B, $n = 0$, at C, $n = 1$, and at D, $n = 2$; for $n = 0$, the formula is, $Q = 199.8\, L\, D\, 3/2$.

Frequently it is of advantage to have the weir plank B below the lower water level, in which case the weir is called a submerged weir. In this case the depths h and H are measured but instead of referring to the table, substitute these measurements in the following equation:

$Q = (C) \times (\text{width of weir in feet}) \times (8.025 \sqrt{a}) \times (h + \frac{2}{3} a)$, in

TABLE I.

h/H	C
.20	.618 to .628
.40	.590 to .600
.60	.583 to .593
.70	.580 to .590
.80	.581 to .591
.90	.590 to .600
.95	.610 to .615

which Q = cubic feet of water per second, $a = H\text{-}h$, and C is a coefficient depending on $h \div H$ (see Table I).

DAMS.

While the flow over the standard weir can be obtained from Table II, which is based on Francis' formula, it has been found that for the forms of crest found on power dams the flow varies a good deal from that for the sharp crested weir. The experiments made at Cornell are the latest and best data we have on the subject, and Figs. 16 to 21 give the coefficients C for six different dams and for depths of water up to six feet as given by these experiments.

It will be seen that the coefficient C varies considerably from 3.33 as used in the Francis formula.

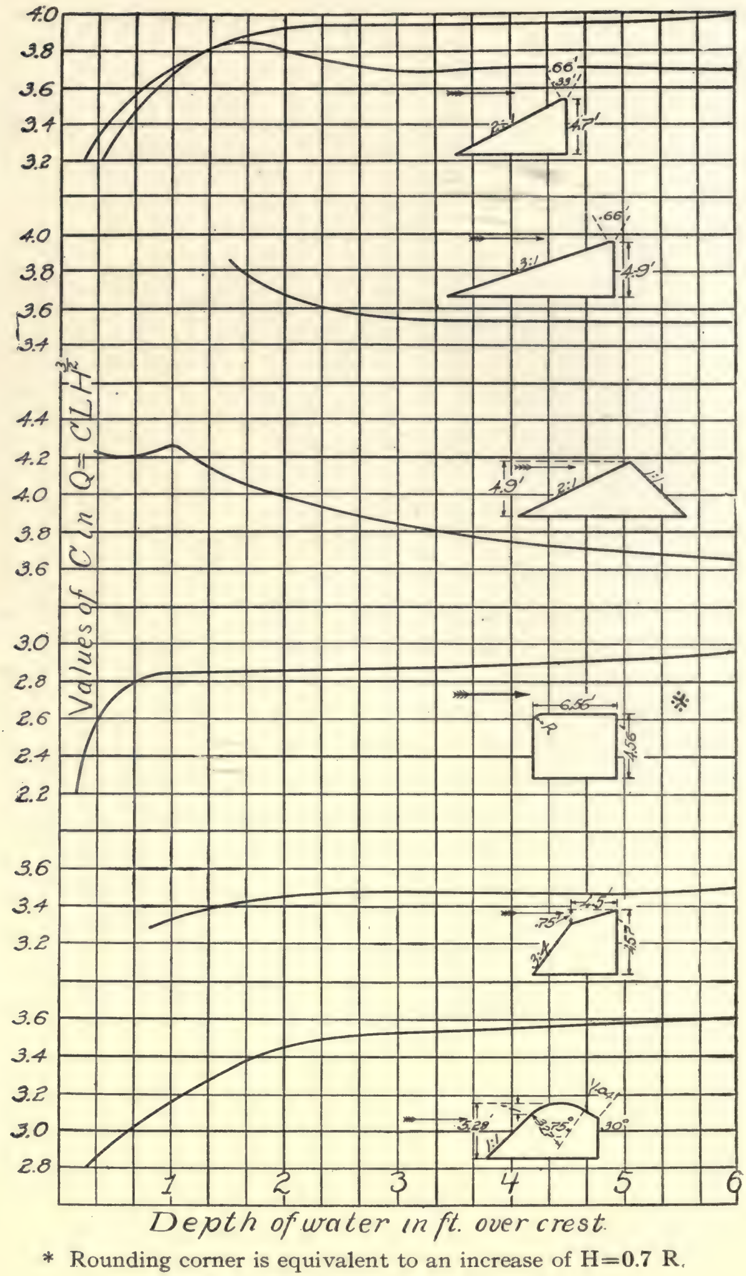
EXAMPLE:—If the depth of water on a dam 230 feet long,

TABLE II.

Weir Table Using Francis' Formula $Q = 3.33\,l\,h^{\frac{3}{2}}$.

Discharge in cubic feet per minute per foot length of Weir.

Head H		Cu. ft. per min.	Head H		Cu. ft. per min.	Head H		Cu. ft. per min.	Head H		Cu. ft. per min.	Head H		Cu. ft. per min.
In Ins.	In ft.		In Ins.	In ft.		In Ins.	In ft.		In Ins.	In ft.		In Ins.	In ft.	
$\frac{1}{8}$	.01	.18	7	.58	88.26	$13\frac{3}{4}$	1.15	246.42	$20\frac{5}{8}$	1.72	450.72	$27\frac{1}{2}$	2.29	692.40
$\frac{1}{4}$	.02	.54	$7\frac{1}{16}$	.59	90.54	$13\frac{15}{16}$	1.16	249.60	$20\frac{3}{4}$	1.73	454.62	$27\frac{5}{8}$	2.30	696.90
$\frac{3}{8}$	.03	1.02	$7\frac{3}{16}$	.60	92.88	$14\frac{1}{16}$	1.17	252.84	$20\frac{7}{8}$	1.74	458.58	$27\frac{3}{4}$	2.31	701.46
$\frac{1}{2}$	.04	1.62	$7\frac{5}{16}$	.61	95.16	$14\frac{1}{8}$	1.18	256.08	21	1.75	462.54	$27\frac{13}{16}$	2.32	706.02
$\frac{5}{8}$	.05	2.22	$7\frac{7}{16}$	.62	97.56	$14\frac{1}{4}$	1.19	259.38	$21\frac{1}{8}$	1.76	466.50	28	2.33	710.58
$\frac{3}{4}$	.06	2.94	$7\frac{9}{16}$	.63	99.90	$14\frac{3}{8}$	1.20	262.62	$21\frac{1}{4}$	1.77	470.52	$28\frac{1}{8}$	2.34	715.20
$\frac{27}{32}$	.07	3.72	$7\frac{11}{16}$	.64	102.30	$14\frac{1}{2}$	1.21	265.92	$21\frac{3}{8}$	1.78	474.48	$28\frac{1}{4}$	2.35	719.76
$\frac{31}{32}$	.08	4.50	$7\frac{13}{16}$	.65	104.70	$14\frac{5}{8}$	1.22	269.22	$21\frac{1}{2}$	1.79	478.50	$28\frac{5}{16}$	2.36	724.38
$1\frac{1}{16}$	.09	5.40	$7\frac{7}{8}$	.66	107.16	$14\frac{3}{4}$	1.23	272.58	$21\frac{5}{8}$	1.80	482.52	$28\frac{7}{16}$	2.37	729.00
$1\frac{3}{16}$	.10	6.30	8	.67	109.56	$14\frac{7}{8}$	1.24	275.88	$21\frac{3}{4}$	1.81	486.54	$28\frac{9}{16}$	2.38	733.62
$1\frac{5}{16}$	.11	7.26	$8\frac{1}{8}$	.68	112.02	15	1.25	279.24	$21\frac{7}{8}$	1.82	490.56	$28\frac{11}{16}$	2.39	738.24
$1\frac{7}{16}$	.12	8.28	$8\frac{1}{4}$	.69	114.54	$15\frac{1}{8}$	1.26	282.60	22	1.83	494.64	$28\frac{13}{16}$	2.40	742.86
$1\frac{1}{2}$	.13	9.36	$8\frac{3}{8}$	.70	117.00	$15\frac{1}{4}$	1.27	285.96	$22\frac{1}{8}$	1.84	498.66	$28\frac{15}{16}$	2.41	747.54
$1\frac{5}{8}$	.14	10.44	$8\frac{1}{2}$	.71	119.52	$15\frac{3}{8}$	1.28	289.32	$22\frac{3}{16}$	1.85	502.74	$29\frac{1}{16}$	2.42	752.16
$1\frac{3}{4}$	.15	11.58	$8\frac{5}{8}$	.72	122.04	$15\frac{1}{2}$	1.29	292.74	$22\frac{5}{16}$	1.86	506.82	$29\frac{3}{16}$	2.43	756.84
$1\frac{15}{16}$	.16	12.78	$8\frac{3}{4}$	.73	124.62	$15\frac{5}{8}$	1.30	296.16	$22\frac{7}{16}$	1.87	510.90	$29\frac{1}{4}$	2.44	761.52
$2\frac{1}{16}$	.17	13.98	$8\frac{7}{8}$	.74	127.20	$15\frac{3}{4}$	1.31	299.58	$22\frac{9}{16}$	1.88	515.04	$29\frac{3}{8}$	2.45	766.20
$2\frac{1}{8}$	.18	15.24	9	.75	129.78	$15\frac{13}{16}$	1.32	303.00	$22\frac{11}{16}$	1.89	519.12	$29\frac{1}{2}$	2.46	770.88
$2\frac{1}{4}$	.19	16.56	$9\frac{1}{8}$	.76	132.36	16	1.33	306.48	$22\frac{13}{16}$	1.90	523.26	$29\frac{5}{8}$	2.47	775.62
$2\frac{3}{8}$	.20	17.88	$9\frac{1}{4}$	.77	135.50	$16\frac{1}{8}$	1.34	309.90	$22\frac{15}{16}$	1.91	527.40	$29\frac{3}{4}$	2.48	780.30
$2\frac{1}{2}$	.21	19.20	$9\frac{3}{8}$	.78	137.64	$16\frac{1}{4}$	1.35	313.38	23	1.92	531.54	$29\frac{7}{8}$	2.49	785.04
$2\frac{5}{8}$	.22	20.64	$9\frac{1}{2}$	.79	140.23	$16\frac{5}{16}$	1.36	316.86	$23\frac{1}{8}$	1.93	535.74	30	2.50	789.78
$2\frac{3}{4}$	.23	22.02	$9\frac{5}{8}$	.80	142.93	$16\frac{7}{16}$	1.37	320.40	$23\frac{1}{4}$	1.94	539.88	$30\frac{1}{8}$	2.51	
$2\frac{7}{8}$	.24	23.52	$9\frac{3}{4}$	.81	145.63	$16\frac{9}{16}$	1.38	323.88	$23\frac{3}{8}$	1.95	544.08	$30\frac{1}{4}$	2.52	
3	.25	24.96	$9\frac{7}{8}$	.82	148.33	$16\frac{11}{16}$	1.39	327.42	$23\frac{1}{2}$	1.96	548.28			
$3\frac{1}{8}$	.26	26.46	10	.83	151.03	$16\frac{13}{16}$	1.40	330.96	$23\frac{5}{8}$	1.97	552.48			
$3\frac{1}{4}$	.27	28.02	$10\frac{1}{8}$	.84	153.84	$16\frac{15}{16}$	1.41	334.50	$23\frac{3}{4}$	1.98	556.68			
$3\frac{3}{8}$	.28	29.58	$10\frac{3}{16}$	.85	156.60	$17\frac{1}{16}$	1.42	338.10	$23\frac{7}{8}$	1.99	560.88			
$3\frac{1}{2}$	.29	31.20	$10\frac{5}{16}$	.86	159.36	$17\frac{3}{16}$	1.43	341.64	24	2.00	564.14			
$3\frac{5}{8}$	.30	32.82	$10\frac{7}{16}$	.87	162.12	$17\frac{1}{4}$	1.44	345.24	$24\frac{1}{8}$	2.01	569.34			
$3\frac{3}{4}$	.31	34.50	$10\frac{9}{16}$	.88	164.94	$17\frac{3}{8}$	1.45	348.84	$24\frac{1}{4}$	2.02	573.60			
$3\frac{13}{16}$	.32	36.18	$10\frac{11}{16}$	.89	167.76	$17\frac{1}{2}$	1.46	352.50	$24\frac{3}{8}$	2.03	577.86			
4	.33	37.86	$10\frac{13}{16}$	.90	170.58	$17\frac{5}{8}$	1.47	356.10	$24\frac{1}{2}$	2.04	582.18			
$4\frac{1}{8}$	.34	39.60	$10\frac{15}{16}$	.91	173.46	$17\frac{3}{4}$	1.48	359.76	$24\frac{5}{8}$	2.05	586.44			
$4\frac{1}{4}$	.35	41.40	11	.92	176.34	$17\frac{7}{8}$	1.49	363.42	$24\frac{3}{4}$	2.06	590.76			
$4\frac{5}{16}$	.36	43.14	$11\frac{1}{8}$	.93	179.22	18	1.50	367.08	$24\frac{27}{32}$	2.07	595.02			
$4\frac{7}{16}$	.37	44.94	$11\frac{1}{4}$	.94	182.10	$18\frac{1}{8}$	1.51	370.74	$24\frac{31}{32}$	2.08	599.34			
$4\frac{9}{16}$	.38	46.81	$11\frac{3}{8}$	.95	184.98	$18\frac{1}{4}$	1.52	374.40	$25\frac{1}{16}$	2.09	603.72			
$4\frac{11}{16}$	.39	48.66	$11\frac{1}{2}$	.96	187.92	$18\frac{3}{8}$	1.53	378.12	$25\frac{3}{16}$	2.10	608.04			
$4\frac{13}{16}$	.40	50.52	$11\frac{5}{8}$	.97	190.86	$18\frac{1}{2}$	1.54	381.84	$25\frac{5}{16}$	2.11	612.36			
$4\frac{15}{16}$	.41	52.44	$11\frac{3}{4}$	.98	193.86	$18\frac{5}{8}$	1.55	385.56	$25\frac{7}{16}$	2.12	616.74			
$5\frac{1}{16}$	.42	54.36	$11\frac{7}{8}$	.99	196.80	$18\frac{3}{4}$	1.56	389.28	$25\frac{1}{2}$	2.13	621.12			
$5\frac{3}{16}$	.43	56.34	12	1.00	199.80	$18\frac{13}{16}$	1.57	393.06	$25\frac{5}{8}$	2.14	625.50			
$5\frac{1}{4}$	.44	58.32	$12\frac{1}{8}$	1.01	202.80	19	1.58	396.78	$25\frac{3}{4}$	2.15	629.88			
$5\frac{3}{8}$	.45	60.30	$12\frac{1}{4}$	1.02	205.80	$19\frac{1}{16}$	1.59	400.56	$25\frac{15}{16}$	2.16	634.26			
$5\frac{1}{2}$	.46	62.34	$12\frac{3}{8}$	1.03	208.86	$19\frac{3}{16}$	1.60	404.34	$26\frac{1}{16}$	2.17	638.70			
$5\frac{5}{8}$	.47	64.38	$12\frac{1}{2}$	1.04	211.92	$19\frac{5}{16}$	1.61	408.18	$26\frac{1}{8}$	2.18	643.08			
$5\frac{3}{4}$	.48	66.42	$12\frac{5}{8}$	1.05	214.98	$19\frac{7}{16}$	1.62	411.96	$26\frac{1}{4}$	2.19	647.52			
$5\frac{7}{8}$	.49	68.52	$12\frac{3}{4}$	1.06	218.04	$19\frac{9}{16}$	1.63	415.80	$26\frac{3}{8}$	2.20	651.96			
6	.50	70.62	$12\frac{27}{32}$	1.07	221.16	$19\frac{11}{16}$	1.64	419.64	$26\frac{1}{2}$	2.21	656.40			
$6\frac{1}{8}$	.51	72.78	$12\frac{31}{32}$	1.08	224.22	$19\frac{13}{16}$	1.65	423.48	$26\frac{5}{8}$	2.22	660.90			
$6\frac{1}{4}$	.52	74.34	$13\frac{1}{16}$	1.09	227.40	$19\frac{7}{8}$	1.66	427.32	$26\frac{3}{4}$	2.23	665.34			
$6\frac{3}{8}$	.53	77.10	$13\frac{3}{16}$	1.10	230.52	20	1.67	431.22	$26\frac{7}{8}$	2.24	669.84			
$6\frac{1}{2}$	.54	79.26	$13\frac{5}{16}$	1.11	233.64	$20\frac{1}{8}$	1.68	435.06	27	2.25	674.34			
$6\frac{5}{8}$	.55	81.48	$13\frac{7}{16}$	1.12	236.82	$20\frac{1}{4}$	1.69	438.96	$27\frac{1}{8}$	2.26	678.84			
$6\frac{3}{4}$	.56	83.70	$13\frac{1}{2}$	1.13	240.00	$20\frac{3}{8}$	1.70	442.86	$27\frac{1}{4}$	2.27	683.34			
$6\frac{13}{16}$	.57	85.98	$13\frac{5}{8}$	1.14	243.18	$20\frac{1}{2}$	1.71	446.76	$27\frac{3}{8}$	2.28	687.84			



* Rounding corner is equivalent to an increase of H=0.7 R.

FIGS. 16 to 21.—Cornell Experiments.

of section shown in Fig. 18, is five feet, what will be the quantity of water passing over the dam?

$$\text{For } h = 5 \text{ we find that } C = 3.7$$
$$Q = C\,l\,h^{\frac{3}{2}} = 3.7 \times 230 \times 5^{\frac{3}{2}}$$

$5^{\frac{3}{2}} = \sqrt{5^3} = 11.18$ and $Q = 9514$ cubic feet per second.

Francis' constant, 3.33, would only have given 8571 cubic feet per second.

Fig. 22, from Cornell experiments shows the form taken by the top surface of the water flowing over four different dam crests and in depths up to six feet.

Fig. 23 shows the under and outer surface of the water pouring

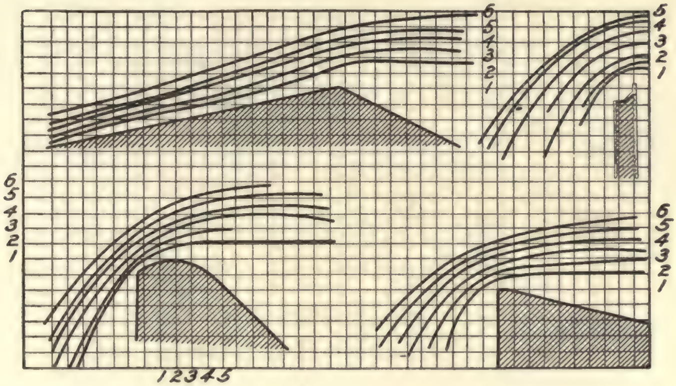


FIG. 22.—Form taken by water passing over a dam (Cornell).

over the dam shown and plotted from measurements made by the author under a gravity dam at Waldron, Ill. During these experiments it was observed that there was a very strong current as indicated at x.

Velocity of Approach.

In most cases met with in actual practice, the water approaches the dam with a greater velocity than that where the formulas here given were evolved, therefore such velocity must be allowed for in applying the formulas.

Let H be the true head of water on the dam (Fig. 24); h = the *observed* head at a distance of six or ten feet above the dam; Q_h = discharge over the dam due to the head h in cubic feet

per second per foot width of weir.; Q = discharge due to the head H; v = velocity of approach in feet per second.

$$v = \sqrt{64.4\ (H-h)} = \frac{Q}{A} \quad \text{(see Fig. 24.)} \quad (1)$$

Then

$$H_v = H - h = \frac{v^2}{64.4} = \left(\frac{Q}{A}\right)^2 \times \frac{1}{64.4} = \text{velocity head.} \quad (2)$$

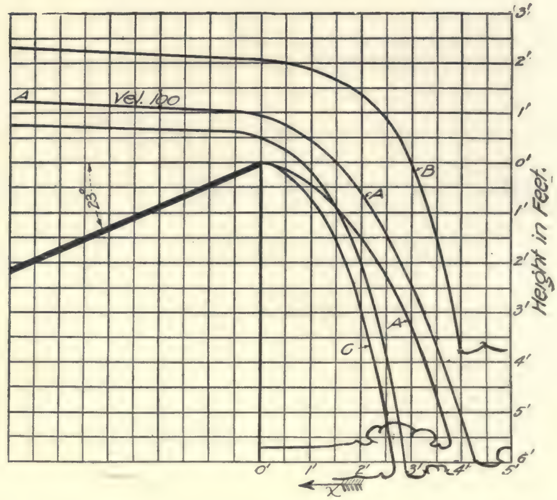


FIG. 23.—Form taken by water passing over a dam (Beardsley).

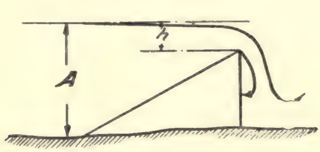


FIG. 24.

To apply these formulas obtain Q from the discharge formula, $Q = C\,l\,h^{\frac{3}{2}}$, and substitute for Q in (1). This gives an approximate value for v, which substituted in $\frac{v^2}{64.4} = H_v$ gives the approximate velocity head, H_v. Then $H_v + h = H$, gives a close approximation to the true head, H, from which

by again substituting in the formula for flow in cubic feet per second, a more nearly exact value for Q can be found. If till greater accuracy is desired the process may be gone through again.

Where possible the value of v should be determined with a meter and substituted direct in $\frac{v}{64.4} = H$.

The Nappe.

In discussing the flow over dams much is said about this and that form of nappe. Nappe is the name given to that part of the crest of the dam which is in direct contact with the water. The various forms of nappe are as follows:

Depressed, wetted, adhering and free.

A depressed nappe is due to the formation of a vacuum; the sheet of water is more or less pressed in upon the crest.

An adhering nappe is partly caused by the vacuum but applies to all cases where there is no air at all between the water and the crest.

Wetted nappe is where the air has free access behind the overpour and the crest is smooth.

Free nappe is where there is no vacuum at all.

The effect on the coefficient C of the different nappes on the same dam is shown by the following experiment on a short low dam.

	C
(1) Free nappe, under surface open to air	4.33—3.47
(2) Depressed nappe, imprisoning a certain amount of air at a pressure below normal	4.60—3.69
(3) Nappe wetted, no air imprisoned; level of tail water at least 4.2 feet below level of crest	4.97—3.99
(4) Adhering nappe	5.54—4.45

These experiments would show a variation of 20 per cent., but as they were for a low and short dam, and as all peculiarities tend to disappear for heavy discharges, in practice the difference would not be so noticeable.

The curves for the six dams in the Cornell experiments are plotted for free nappes.

It will be seen that the gravity dam will pass more water

for a given depth of water on the crest than will a thin edged weir, or any other form of dam.

VENTURI METERS.

Next to the standard weir, the Venturi meter is the most important aid in the measurement of water, and when properly installed and calibrated it excels the weir in accuracy.

Many of the largest cities of the country are using Venturi meters to measure the flow of water in the water systems, and in a few instances such meters are being used to measure the flow to turbines. About 1887, Mr. Clemens Herschel brought the Venturi to the attention of American engineers and since that time it has been steadily growing in favor.

Fig. 25 shows the proportions (all dimensions in feet) for the Venturi meter used by Mr. Herschel. When used for

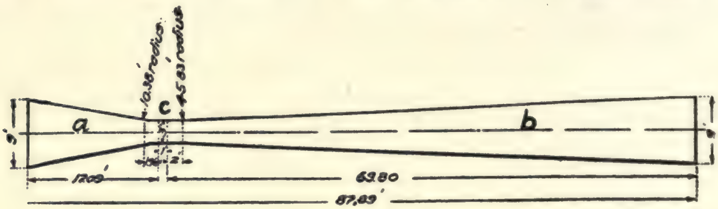


FIG. 25.

head gates, however, (see Fig. 26) the meter may be materially shortened.

Fig. 26 shows the Venturi meter for measuring the flow to a turbine serving the double purpose of a meter and a head gate. Mr. Herschel states that a Venturi meter placed in a nine-foot penstock in which the mean velocity is about 2.5 to three feet per second, the total loss in head will be one foot and if the velocity is two feet per second the loss will be only six inches. Therefore in all but water powers of the lowest head the Venturi meter can be used to measure the water. Since it does not affect the accuracy of the meter if the cones, *a* and *b* (Fig. 25), are rough, these parts may be constructed of reinforced concrete. It is essential, however, that the throat *c* be made accurately and of metal lined with bronze or brass. The Builders' Iron Foundry of Providence, R. I., make a specialty of Venturi meters. The air chambers *D* and *E*, Fig. 26,

must also be of metal. Fig. 27 shows one form of air chamber. The effect would, however, be just as satisfactory if the air space was not divided into segments and if but a single pipe was inserted at the top.

About the only condition that effects the efficiency of the Venturi is the falling off in the velocity of the water when the demand is low. This would not be an objection where used for turbines, as the flow would never be below about 20 per cent. of the full flow. Mr. Frizell states that the area at the throat must be more than 1/16 to 1/20 of the area of the penstock at *C* Fig. 26, in order that the meter should accurately measure

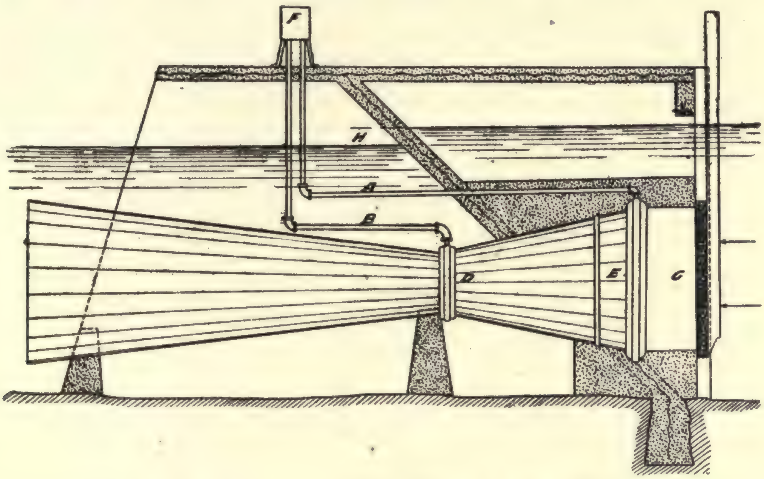


FIG. 26.—Venturi meter used as a head gate.

the flow. Herschel says that the velocity through the throat must not be less than five feet per second. This limits the shortening of the meter as the taper of the cones meeting at the throat must be as given in Fig. 25. When used for head gates, as in Fig. 26, several Venturi meters should be placed side by side, in which case if all the turbines are running at part gate, one or more meters can be cut out and the remaining meters used to their full capacity and efficiency. When so arranged the meters may be made comparatively short. Nothing can clog up a Venturi meter even saw logs will be metered as water. Mr. Herschel claims that the Venturi meter is accurate to within about two per cent.

The pipes A and B, Fig. 26, lead up to the indicating instrument F. This instrument is made in various forms by the Builders' Iron Foundry and gives the flow in cubic feet per minute.

Penstocks and Pipes.

We are indebted to Kutter and D'Arcy for our most reliable data on the flow of water through pipes; they performed many experiments with pipes of various sizes and lengths, and their formulas may be depended on as being exact to within five per cent. We will only give one formula, and the accompanying tables, the author's purpose being to give only the best and easiest, rather than to give a variety.

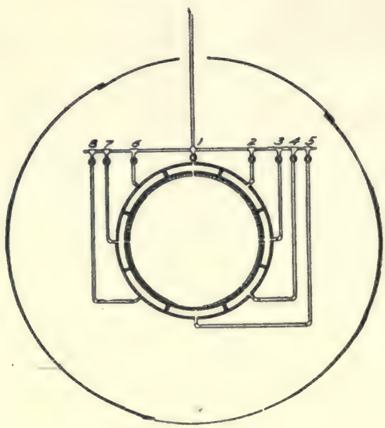


Fig. 27

$Q = A\,C\sqrt{r} \times \sqrt{s} \quad v = C\sqrt{r\,s} = C\sqrt{r} \times \sqrt{r}$

v = velocity of water in feet per second.

A = wet area of penstock = 96, Fig. 29.

C = a coefficient depending on s, n, r,

s = the fall of water surface per foot length or hydraulic gradient

$$= \frac{\text{fall in feet per mile}}{5280}$$

$r = \dfrac{A}{P}$ = mean hydraulic depth,

P = wetted perimeter (see Figs. 28-30).

TABLE III (Kent).

Q = discharge in cubic feet per second, A = area in square feet, v = velocity in feet per second, r = mean hydraulic depth, ¼ diam. for pipes running full, s = sine of slope.

Size of Pipe			Clean Cast-iron Pipes.		Value of $AC\sqrt{r}$ by Kutter's Formula, when n = .013.	Old Cast-iron Pipes Lined with Deposit.	
d = diam. in ft.	in.	A = area in square feet.	For Velocity, $C\sqrt{r}$.	For Discharge, $AC\sqrt{r}$.		For Velocity. $C\sqrt{r}$.	For Discharge $AC\sqrt{r}$.
	⅜	.00077	5.251	.00403		3.532	.00272
	½	.00136	6.702	.00914		4.507	.00613
	¾	.00307	9.309	.02855		6.261	.01922
	1	.00545	11.61	.06334		7.811	.04257
	1¼	.00852	13.68	.11659		9.255	.07885
	1½	.01227	15.58	.19115		10.48	.12855
	1¾	.01670	17.32	.28936		11.65	.19462
	2	.02182	18.96	.41357		12.75	.27824
	2½	.0341	21.94	.74786		14.76	.50321
	3	.0491	24.63	1.2089		16.56	.81333
	4	.0873	29.37	2.5630		19.75	1.7246
	5	.136	33.54	4.5610		22.56	3.0681
	6	.196	37.28	7.3068	4.822	25.07	4.9147
	7	.267	40.65	10.852		27.34	7.2995
	8	.349	43.75	15.270		29.43	10.271
	9	.442	46.73	20.652	15.03	31.42	13.891
	10	.545	49.45	26.952		33.26	18.129
	11	.660	52.16	34.428		35.09	23.158
1		.785	54.05	42.918	33.50	36.75	28.867
1	2	1.000	59.34	63.435		39.91	42.668
1	4	1.396	63.67	88.886		42.83	59.788
1	6	1.767	67.75	119.72	102.14	45.57	80.531
1	8	2.182	71.71	156.46		48.34	105.25
1	10	2.640	75.32	198.83		50.658	133.74
2		3.142	78.80	247.57	224.63	52.961	166.41
2	2	3.687	82.15	302.90		55.258	203.74
2	4	4.276	85.39	365.14		57.436	245.60
2	6	4.909	88.39	433.92	411.37	59.455	291.87
2	8	5.585	91.51	511.10		61.55	343.8
2	10	6.305	94.40	595.17		63.49	400.3
3		7.068	97.17	686.76	674.09	65.35	461.9
3	2	7.875	99.93	786.94		67.21	529.3
3	4	8.726	102.6	895.7		69.	602.
3	6	9.621	105.1	1011.2	1021.1	70.70	680.2
3	8	10.559	107.6	1136.5		72.40	764.5
3	10	11.541	110.2	1271.4		74.10	855.2
4		12.566	112.6	1414.7	1463.9	75.73	951.6
4	3	14.186	116.1	1647.6		78.12	1108.2
4	6	15.904	119.6	1901.9	2007.	80.43	1279.2
4	9	17.721	122.8	2176.1		82.20	1456.8
5		19.635	126.1	2476.4	2659.	84.83	1665.7
5	3	21.648	129.3	2799.7		86.99	1883.2
5	6	23.758	132.4	3146.3	3429.	89.07	2116.2
5	9	25.967	135.4	3516.		91.08	2365.
6		28.274	138.4	3912.8	4322.	93.08	2631.7
6	6	33.183	144.1	4782.1	5339.	96.93	3216.4
7		38.485	149.6	5757.5	6510.	100.61	3872.5
7	6	44.179	154.9	6841.6	7814.	104.11	4601.9
8		50.266	160.	8043.	9272.	107.61	5409.9
8	6	56.745	165.	9364.7	10889.	111.	6299.1
9		63.617	169.8	10804.	12663.	114.2	7267.3
9	6	70.882	174.5	12370.	14597.	117.4	8320.6
10		78.540	179.1	14066.	16709.	120.4	9460.9

n = the coefficient of roughness.
Q = cubic feet of water per second

$$r \text{ in Fig. 28} = \frac{6\times 12}{6+12+6}$$

$$r \text{ in Fig. 29} = \frac{8\times 12}{8+12+8+12}$$

Established Values of n.

In all formulas for flow of water in penstocks the one uncertain factor is n; the value of n for any particular pipe must be guessed at before the capacity of the pipe can be determined. The following will be useful in forming a close guess. It is always the best policy to take the next higher coefficient than the one which seems to fit the case.

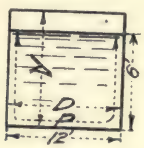


Fig. 28.

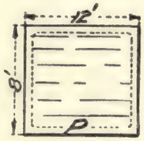


Fig. 29.

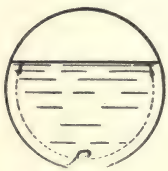


Fig. 30.

n = .01: This value has usually been used for wood stave pipes, concrete, etc., but has been found too small in practice.

n = .012: plaster of pure cement; planed timber, glazed, coated or enameled stoneware and iron pipes; glazed surfaces of every sort and all built in good order, and where it can be cleaned and pipes constantly full.

n = .013: unplaned timber when all joints are good, such as should be the case in all penstocks full of water at all times.

n = .0135: ashlar masonry and well-laid brickwork, riveted steel pipes, common cast-iron pipes after a year's use, earthen and stoneware pipe in good condition but not new; plaster, and planed wood in bad condition, and generally the materials mentioned with n = .012 when same is inaccessible and high velocities.

n = .015: second class or rough faced brickwork, well dressed stonework, foul and slightly tuberculated iron, cement and terra cotta pipes with bad joints, and old; high velocities.

n = .017: brickwork, ashlar and stoneware in bad condition, tuberculated iron pipes, rubble in cement or plaster in good order, fine gravel well rammed, of one-third to two-thirds inches diameter; and generally the materials mentioned for n = .013 when same are in bad order, and low velocities used.

n = .02: rubble in cement in bad condition, coarse rubble, rough set and undressed; coarse rubble set dry; disintegrated brickwork and masonry; coarse gravel well rammed from 1to1½ inches in diameter; canals with beds and banks of very firm regular gravel carefully trimmed and rammed, rough rubble with bed partly covered with silt and mud, penstocks made of rough narrow lumber and with battens over cracks, trimmed earth canals.

n = .0225: canals in earth in first-class condition.

n = .025: canals and rivers in earth of fairly uniform cross-section, slope and direction in good shape and free from loose stones and weeds.

n = .0275: canals and rivers below the average in order and regimen.

n = .03: canals and rivers in earth in rather bad order and regimen, having loose stones and occasional weeds and detritus

n = .035: rivers and canals with earthen beds in bad order and regimen and having many boulders, snags, weeds, bends, ice, etc.

n = .055: torrential streams of high velocity filled with boulders, etc.

U. S. engineers found that for the Susquehanna River during the great flood of 1904 when the river was out of its banks and running through timbered land, over islands, etc., that n = .055.

CIRCULAR PENSTOCKS.

We now have the data necessary to find the quantity, Q. The first step is to select the coefficient n, and from Table IV find the corresponding value of $A\ C\sqrt{r}$ for the particular pipe under consideration. Then from Table V the value of $\sqrt{s}$ is found which corresponds to the pipe and fall per mile under consideration. These values substituted in the formula $Q = \sqrt{s} \times A\ C\sqrt{r}$ give us the flow in cubic feet per second.

*The above values have been made to agree with latest experiments.

TABLE IV. (Kent)

FOR CIRCULAR PIPES FLOWING FULL. VALUES OF $A\,C\,\sqrt{r}$ IN KUTTER'S FORMULA.

$$Q = AC\sqrt{r}\,\sqrt{s}$$

Diam. ft. in.	Value of $AC\sqrt{r}$.					
	n = .010	n = .011	n = .012	n = .013	n = .015	n = .017
6	6.906	6.0627	5.3800	4.8216	3.9640	3.329
9	21.25	18.742	16.708	15.029	12.421	10.50
1 0	46.93	41.487	37.149	33.497	27.803	23.60
1 3	86.05	76.347	68.44	61.867	51.600	43.93
1 6	141.2	125.60	112.79	102.14	85.496	72.99
1 9	214.1	190.79	17 .66	155.68	130.58	111.8
2	307.6	274.50	247.33	224.63	188.77	164.
2 3	421.9	377.07	340.10	309.23	260.47	223.9
2 6	559.6	500.78	452.07	411.27	347.28	299.3
2 9	722.4	647.18	584.90	532.76	451.23	388.8
3	911.8	817.50	739.59	674.09	570.90	493.3
3 3	1128.9	1013.1	917.41	836.69	709.56	613.9
3 6	1374.7	1234.4	1118.6	1021.1	866.91	750.8
3 9	1652.1	1484.2	1345.9	1229.7	1045.	906.
4	1962.8	1764.3	1600.9	1463.9	1245.3	1080.7
4 6	2682.1	2413.3	2193.	2007.	1711.4	1487.3
5	3543.	3191.8	2903.6	2659.	2272.7	1977.
5 6	4557.8	4111.9	3742.7	3429.	2934.8	2557.2
6	5731.5	5176.3	4713.9	4322.	3702.3	3232.5
6 6	7075.2	6394.9	5825.9	5339.	4588.3	4010.
7	8595.1	7774.3	7087.	6510.	5591.6	4893.
7 6	10296.	9318.3	8501.8	7814.	6717.	5884.2
8	12196.	11044.	10083.	9272.	7978.3	6995.3
8 6	14298.	12954.	11832.	10889.	9377.9	8226.3
9	16604.	15049.	13751.	12663.	10917.	9580.7
9 6	19118.	17338.	15847.	14597.	12594.	11061.
10	21858.	19834.	18134.	16709.	14426.	12678.
10 6	24823.	22534.	20612.	18996.	16412.	14434.
11	28020.	25444.	23285.	21464.	18555.	16333.
11 6	31482.	28593.	26179.	24139.	20879.	18395.
12	35156.	31937.	29254.	26981.	23352.	20584.
12 6	39104.	35529.	32558.	30041.	26012.	22938.
13	43307.	39358.	36077.	33301.	28850.	25451.
13 6	47751.	43412.	39802.	36752.	31860.	28117.
14	52491.	47739.	43773.	40432.	35073.	30965.
14 6	57496.	52308.	47969.	44322.	38454.	33975.
15	62748.	57103.	52382.	48413.	42040.	37147.
16	74191.	67557.	62008.	57343.	49823.	44073.
17	86769.	79050.	72594.	67140.	58387.	51669.
18	100617.	91711.	84247.	77932.	67839.	60067.
19	115769.	105570.	96991.	89759.	78201.	69301.
20	132133.	120570.	110905.	102559.	89423.	79259.

EXAMPLE: It is desired to carry 60,000 cubic feet of water per minute a distance of six miles with a loss of head of 6 feet. A wooden stave pipe built of planed staves is to be used.

Under these conditions we should select n as = to .01. From Table IV we find that for n = .01 and for a pipe 14 feet in

diameter (trial size), $A\ C\sqrt{r} = 52{,}491$ and from Table V we find that for a fall of one foot per mile $\sqrt{s} = .013762$. Therefore $Q = .013762 \times 52{,}491 = 722.38$ cubic feet per second, or 43,342 cubic feet per minute. This is not a large enough flow, so we next try a 16 foot pipe for which $A\ C\sqrt{r} = 74{,}191$ and $\sqrt{s} = .013762$ and $A\ C\sqrt{r}\sqrt{s} = 61{,}260$ cubic feet per minute.

Knowing the size of pipe and the quantity of water flowing

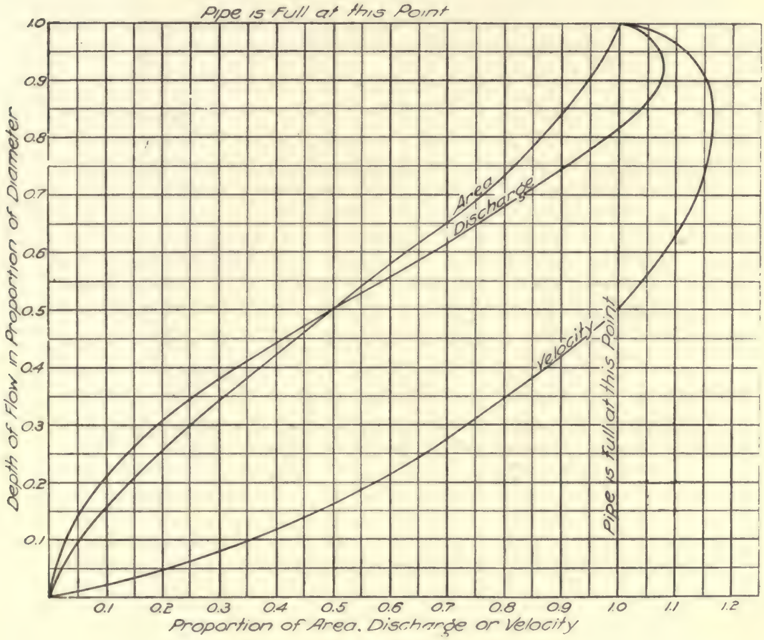


Fig. 31.

the velocity is obtained by dividing the quantity Q, by the area of the pipe in square feet.

The velocity may also be obtained from the curves, Fig. 31, and it is a good policy to use both methods as a check on the calculations.

Table V will be found useful in getting the square roots of the various values of s.

Flow in Penstocks.

We have Kutter's formula, $v = C\sqrt{r\,s}$ where

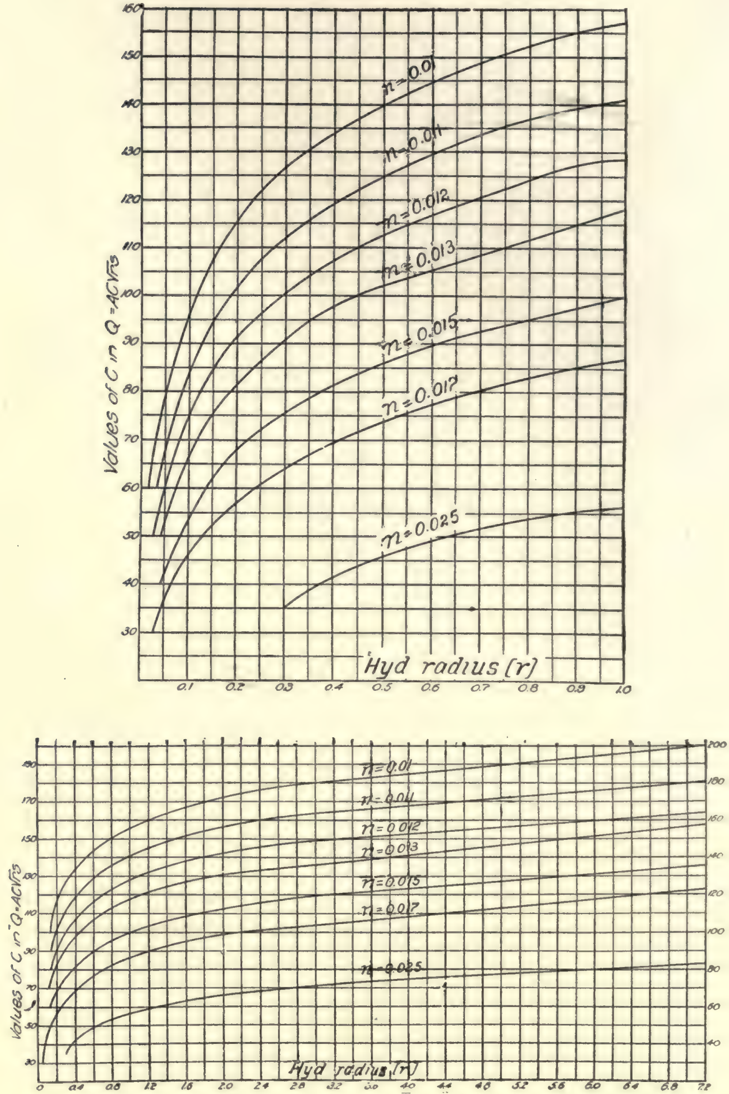

Values of C in Q = AC√rs
n = 0.01
n = 0.011
n = 0.012
n = 0.013
n = 0.015
n = 0.017
n = 0.025
Hyd radius (r)
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
30 40 50 60 70 80 90 100 110 120 130 140 150 160
Values of C in Q = AC√rs
n = 0.01
n = 0.011
n = 0.012
n = 0.013
n = 0.015
n = 0.017
n = 0.025
Hyd radius (r)
0 0.4 0.8 1.2 1.6 2.0 2.4 2.8 3.2 4.0 4.4 4.8 5.2 5.6 6.0 6.4 6.8 7.2
40 60 80 100 120 140 160 180 200

Fig. 32.

$$C = +\frac{23 + \frac{1}{n} + \frac{.00155}{s}}{.5521 + \left[23 + \frac{.00155}{s}\right]\frac{n}{\sqrt{r}}}$$

This is a rather laborious equation and in Fig. 32 is given a set of curves from which C may be easily found. Each curve is for a certain value of n as given on page 22 and a corresponding hydraulic mean radius, r.

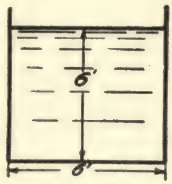


FIG. 33.

These curves give very close results for all slopes greater than $s = .0005$ or 3 feet per mile, and pipes of greater diameter than 10 inches. For smaller slopes or penstocks the curves give fairly approximate values.

EXAMPLE.—20,000 cubic feet of water per minute is to be carried one mile with a fall of 5 feet. What is the proper size

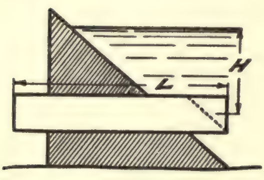


FIG. 34.

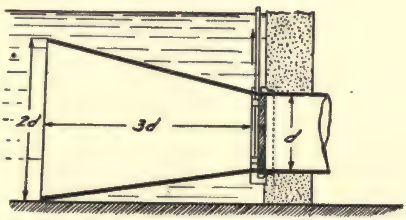


FIG. 35.

for the penstock if built of planed lumber and well constructed?

For a trial size assume a section as shown in Fig. 33, $s = .000947$, $n = .01$, $r = (6\times6) \div (6+6+6) = 2$.

Now from curve for $n = .01$, Fig. 32, and $r = 2$, we find that $C = 172$. Substituting these values in $v = C\sqrt{r\,s}$ we have $v = 172\sqrt{2\times.000947} = 7.5026$ feet per second, or 450 feet per

minute. As the area of the penstock = 36 square feet, cubic feet per minute = 450×36 = 16,200 which is not large enough, so we take a section, say, 7×7 = feet.

Then $r = 2.33$ and $C = 175$. $175 \times \sqrt{r\,s} = v = 175 \times .047$,

TABLE V. (Kent).

FALL IN FEET PER MILE, SLOPE, SINE OF SLOPE AND SQUARE ROOT OF THE SINE.

Fall in ft. per mile.	Slope 1 ft. in-	Sine of angle of slope		Fall in ft. per mile.	Slope 1 ft. in-	Sine of angle of slope.	
H	L	$S = \frac{H}{L}$	$\sqrt{s}$	H	L	S	$\sqrt{s}$
0.25	21120.	.0000473	.006881	17.	310.6	.0032197	.056742
.30	17600.	.0000568	.007538	18.	293.3	.0034091	.058388
.40	13200.	.0000758	.008704	19.	277.9	.0035985	.059988
.50	10560.	.0000947	.009731	20.	264.	.0037879	.061546
.60	8800.	.0001136	.010660	22	240.	.0041667	.064549
.702	7520.	.0001330	.011532	24.	220.	.0045455	.067419
.805	6560.	.0001524	.012347	26.	203.1	.0049242	.070173
.904	5840.	.0001712	.013085	28.	188.6	.0053030	.072822
1.	5280.	.0001894	.013762	30.	176.	.0056818	.075378
1.25	4224.	.0002367	.015386	35.20	150.	.0066667	.081650
1.5	3520.	.0002841	.016854	40.	132.	.0075758	.087039
1.75	3017.	.0003314	.018205	44.	120.	.0083333	.091287
2.	2640.	.0003788	.019463	48.	110.	.0090909	.095346
2.25	2347.	.0004261	.020641	52.8	100.	.010	.1
2.5	2112.	.0004735	.021760	60.	88.	.0113636	.1066
2.75	1920.	.0005208	.022822	66.	80.	.0125	.111803
3.	1760.	.0005682	.023837	70.4	75.	.0133333	.115470
3.25	1625.	.0006154	.024807	80.	66.	.0151515	.123091
3.5	1508.	.0006631	.025751	88.	60.	.0166667	.1291
3.75	1408.	.0007102	.026650	96.	55.	.0181818	.134839
4.	1320.	.0007576	.027524	105.6	50.	.02	.141241
5.	1056.	.0009470	.030773	120.	44.	.0227273	.150756
6.	880.	.0011364	.03371	132.	40.	.025	.158 14
7.	754.3	.0013257	.036416	160.	33.	.0303030	.174077
8.	660.	.0015152	.038925	220.	24.	.0416667	.204124
9.	586.6	.0017044	.041286	264.	20.	.05	.223607
10.	528.	.0018939	.043519	330.	16.	.0625	.25
11.	443.6	.0020833	.045643	440.	12.	.0833333	.288675
12.	440.	.0022727	.047673	528.	10.	.1	.316228
13.	406.1	.0024621	.04962	660.0	8.	.125	.353553
14.	377.1	.0026515	.051493	880.	6.	.1666667	.408248
15.	352.	.0028409	.0533	1056.	5.	.2	.447214
16.	330.	.0030303	.055048	1320.	4.	.25	.5

which gives a velocity of 8.225 feet per second, and a flow of 24,180 cubic feet per minute, which is too much. Another trial gives a section of 6 feet 4 inches $\times$ 7 feet as having a capacity of 21,360 cubic feet per minute, which is about right.

SHORT PIPES.

The flow through such pipes is affected by three quantities; frictional resistance such as has been already considered, losses due to setting quiet water into motion and a loss due to the shape of the orifice.

TABLE VI (Kent).

Values of $\sqrt{r}$ for circular pipes, sewers, and penstocks of different diameters. r = mean hydraulic depth $= \dfrac{\text{area}}{\text{perimeter}} = \frac{1}{4}$ diameter for circular pipes running full or exactly half full.

Diam. ft.	in.	$\sqrt{r}$ in ft.	Diam. ft.	in.	$\sqrt{r}$ in ft.	Diam. ft.	in.	$\sqrt{r}$ in ft.	Diam. ft.	in.	$\sqrt{r}$ in ft.
	⅜	.088	2		.707	4	6	1.061	9		1.500
	½	.102	2	1	.722	4	7	1.070	9	3	1.521
	¾	.125	2	2	.736	4	8	1.080	9	6	1.541
	1	.144	2	3	.750	4	9	1.089	9	9	1.561
	1¼	.161	2	4	.764	4	10	1.099	10		1.581
	1½	.177	2	5	.777	4	11	1.109	10	3	1.601
	1¾	.191	2	6	.790	5		1.118	10	6	1.620
	2	.204	2	7	.804	5	1	1.127	10	9	1.639
	2½	.228	2	8	.817	5	2	1.137	11		1.658
	3	.251	2	9	.829	5	3	1.146	11	3	1.677
	4	.290	2	10	.842	5	4	1.155	11	6	1.696
	5	.323	2	11	.854	5	5	1.164	11	9	1.714
	6	.354	3		.866	5	6	1.173	12		1.732
	7	.382	3	1	.878	5	7	1.181	12	3	1.750
	8	.408	3	2	.890	5	8	1.190	12	6	1.768
	9	.433	3	3	.901	5	9	1.199	12	9	1.785
	10	.456	3	4	.913	5	10	1.208	13		1.083
	11	.479	3	5	.924	5	11	1.216	13	3	1.820
1		.500	3	6	.935	6		1.225	13	6	1.837
1	1	.520	3	7	.946	6	3	1.250	14		1.871
1	2	.540	3	8	.957	6	6	1.275	14	6	1.904
1	3	.559	3	9	.968	6	9	1.299	15		1.936
1	4	.577	3	10	.979	7		1.323	15	6	1.968
1	5	.595	3	11	.990	7	3	1.346	16		2.
1	6	.612	4		1.	7	6	1.369	16	6	2.031
1	7	.629	4	1	1.010	7	9	1.392	17		2.061
1	8	.646	4	2	1.021	8		1.414	17	6	2.091
1	9	.661	4	3	1.031	8	3	1.436	18		2.121
1	10	.677	4	4	1.041	8	6	1.458	19		2.180
1	11	.692	4	5	1.051	8	9	1.479	20		2.236

To find the diameter D of a short pipe of length L, which is to carry a given quantity Q of water per minute under a head H, substitute values for D in the formula

$$D = 0.251 \sqrt[5]{\frac{Q^2}{H}(37.6\,D + L)}$$

until the equation is satisfied.

If the orifice is given a slant as per dotted line (Fig. 34) the flow will be materially reduced.

The inlet on short penstocks where the entrance loss would be an important portion of the whole lost head, should be provided with a conical entrance piece as in Fig. 35; to avoid a large gate, the cone may project out into the head water as shown.

Flow of Air in Pipes.

Air flows in a pipe under the same laws as water in a penstock, the only difference being in the coefficient of friction.

$$H = \frac{v^2 L}{10{,}000\, D^5 d}$$

wherein D is the diameter of the pipe in inches, L the length of the pipe in feet, V the volume of air delivered in cubic feet per minute, H the pressure lost in transmission, and d a constant.

A hydro-compressor delivers the air to the pipe line at the temperature of the water and, therefore, does not require cooling. In other words, the pressure will not drop off due to cooling while being transmitted through the pipes.

TABLE VII.

Limiting Velocities for Air.

Diameter of pipe in feet	2	4	6	8	10	12
Velocity in feet per second	18	12	11	10	9	8

The above velocities are extreme and should be avoided.

TABLE VIII (F. Richard).

α for Wrought Iron Pipe.

Diameter Pipe in inches.	α	Diameter Pipe in inches.	α
1	.35	5	0.934
1¼	.50	6	1.00
1½	.662	8	1.125
2	.665	10	1.2
2½	.65	12	1.26
3	.73	16	1.34
3½	.787	20	1.40
4	.84	24	1.45

TABLE IX (F. Richard).

POWER OBTAINABLE FROM ONE POUND OF COMPRESSED AIR USED WITHOUT HEATING

Pressure lbs. per sq. in.		H.p. per cubic ft. of air per sec.
Of Compression.	After cooling to 60° F.	
100	57.34	56.6
95	55.30	55.4
90	53.17	54.0
85	51.11	52.7
80	48.96	51.2
75	46.59	49.2
70	44.53	47.7
65	42.24	45.8
60	39.92	43.7
55	37.52	41.4
50	35.06	38.7
45	32.58	35.8
40	29.94	32.4
35	27.21	28.4
30	24.39	23.7

CANALS.

The calculations for the flow in canals are even more complicated than for penstocks on account of the greater variation of n. This coefficient varies for each particular soil and lining and may equal .012 for one section and .035 for the next. In selecting the value for n always take a value .01 larger than called for in the list given on page 22. The form of cross-section depends principally on the soil through which the canal is cut. The velocity is selected so that the linings will not be disturbed by the flowing water and sufficient velocity given to the flow so that silt will not be deposited.

The banks of the canal must be made so flat that when they are *wet* they will not run down into the canal or change their form in any way.

Grass will form in most canals, but this can be mowed out and need not be considered.

TABLE X.

SAFE VELOCITIES FOR CANALS, AT WHICH THE VARIOUS SOILS WILL NOT WASH.

Soft brown earth.........	Safe mean velocity	ft.	per	sec.,	0.328
Soft loam..............	"	"	"	"	.656
Sand..................	"	"	"	"	1.312
Gravel.................	"	"	"	"	2.625
Pebbles (most gravel)....	"	"	"	"	3.938
Broken stones, flint......	"	"	"	"	5.579
Conglomerate, soft slate..	"	"	"	"	6.564
Pure clay...............	"	"	"	"	7.000
Stratified rock...........	"	"	"	"	8.204
Hard rock...............	"	"	"	"	13.127

These values are recommended by Kutter as being safe.

EFFECT OF ICE ON THE FLOW.

In all northern latitudes, due allowance must be made for the ice forming in the canal, which not only reduces the area

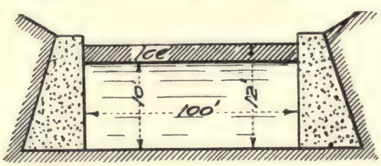


FIG. 36.

but also increases the wetted perimeter. The coefficient n for the surface of the ice may be assumed to be equal to that for a canal in earth or .024.

EXAMPLE.—Find the difference between the capacities of the canal shown in Fig. 36 having a fall of one foot to the mile when free from ice, and when frozen over as shown. (1) Canal free from ice: The area, A, = 1200 square feet, $r = \frac{1200}{124} =$ 9.68. $s = \frac{1}{5280} = .000189$. $n = .024$.

From Kutter's formula

$$v = \frac{23 + \frac{1}{n} + \frac{.00155}{s}}{.5521 + \left[23 + \frac{.00155}{s}\right]\frac{n}{\sqrt{r}}} \times \sqrt{r\,s}$$

Substituting

$$v = \frac{23 + \frac{1}{.024} + \frac{.00155}{.000189}}{.5521 + \left[23 + \frac{.00155}{.000189}\right]\frac{.024}{\sqrt{9.68}}} \times \sqrt{9.68 \times .000189} =$$

$91.7 \times .0428 = 3.92$ feet per second, and $Q = 1200 \times 3.92 = 4700$ cubic feet per second.

(2) Canal frozen over with ice 24 inches thick.

$$A = 1000. \quad r = \frac{1000}{220} = 4.545, \; n = .024$$

Solving for v, $v = 2.31$ and $Q = 1000 \times 2.31 = 2310$.

Thus it will be seen that the ice causes a loss in the efficiency of the canal of about 49 per cent. In the case of a shallow, broad canal with rough bottom, the loss is proportionately greater.

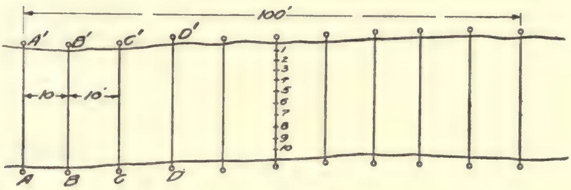


FIG. 37.

Therefore, in selecting the size of section do not fail to allow for ice.

RIVERS, PRELIMINARY MEASUREMENTS.

As most frequently happens the stream to be measured is too wide and deep to warrant the construction of a weir, in which case some other method must be adopted.

Select some place along the river where for from 50 to 100 feet the water is of uniform depth and width, and measure off along the banks a certain distance, say 100 feet, as in Fig. 37. Divide this distance into 10-foot lengths, and mark the divisions with stakes. Take a dry piece of wood and weight one end so that when thrown into the water it will stand almost upright, and, as it floats, just clear the bottom of the stream. A piece of lead pipe is a handy thing for this purpose as it may be cut to any length, and easily nailed to the float.

Have an assistant on the bank with a watch and note book. Throw the float into the stream above the first stake and when it has floated down even with it, the assistant takes the time. Then run down and stand ready to catch the float when it gets even with the last stake, and as it arrives at that point call out and the assistant catches the time. The number of seconds it has taken the float to move 100 feet is then entered in the note book. A large number of these readings should be taken, and the float thrown in at different places across the stream so as to get the average velocity of the water. All these measurements added up and divided by their number, now gives the average time it takes the float to move 100 feet. The average velocity of the river will be from 85 to 95 per cent. of this, depending on the unevenness of the river bed.

Now take a rod divided into feet and inches, and, starting across the stream even with the first stake, measure the depth

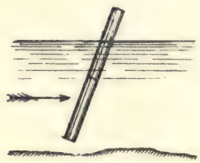


Fig. 38.

every three or four feet, as at 1, 2, 3, 4, etc., being careful to set the rod on top of the inequalities rather than down in between them. Repeat this operation across from every one of the ten stakes and then add up all the soundings and divide the sum by the number of the readings taken. This gives the average depth in feet. Next get the width of the stream opposite each stake, as $A\ A'$, $B\ B'$, etc., and divide by the number of measurements, getting the average width.

Now to get the cubic feet of water flowing per minute multiply the average width by the average depth and this product by the velocity of the water in feet per minute.

A cubic foot of water weighs slightly more than $62\frac{1}{3}$ pounds. Therefore the number of cubic feet of water found to be flowing each minute multiplied by $62\frac{1}{3}$ ($62\frac{1}{2}$ is generally used), gives the pounds of water flowing each minute.

The writer has found that in this way remarkably close re-

sults can be obtained, not varying more than five per cent. from measurements made with a standard weir.

One of the most reliable floats consists of a jointed tube closed at one end. Fig. 38. The joints are short enough to easily pack away and about 2 inches in diameter. In making the measurements enough joints are screwed together so that when sufficient shot is put in to sink the bottom to within a few inches of the river bed, the top is just out of water. Current meters are often used for measuring the velocity of a stream.

Current Meters.

Fig. 39 shows the two types of current meters which are commonly used. The one at the right, is of the cup vane

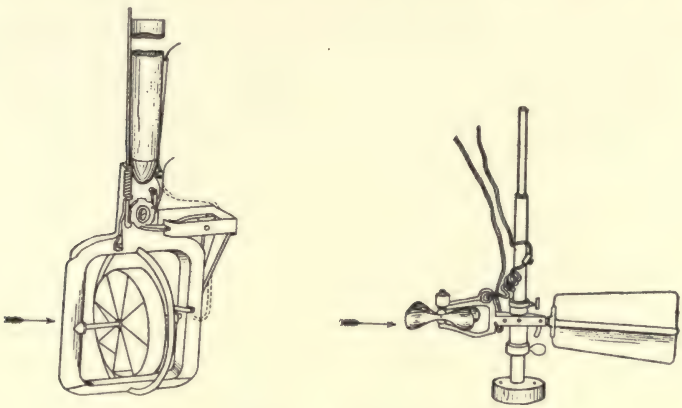


Fig. 39.—Revolving type current-meter.

type, and the one at the left of the helical. In each case the meter is mounted on a long pole or rod and lowered into the water. The current causes the vanes to revolve and the instrument is so calibrated that a certain number of revolutions of the vanes indicates a certain velocity of the water. By proper gearing one of the gears is made to revolve once for each passing foot of water. The revolution of this gear is made known to the observer either by electrically ringing a bell or by causing a click which may be heard along the rod.

It will be seen that these instruments depend for their accuracy upon the constancy of the coefficient of friction of the

numerous bearings. These bearings are in agate mostly and yet a small piece of river grass or a grain cf sand can cause a great inaccuracy in the reading. On this account they must be frequently inspected, cleaned and re-calibrated.

It was to avoid, as much as possible, these sources of error, that the author designed the current meter shown in Fig. 40. In this meter the current strikes the vane, causing it to rotate on the pivot *g*; attached to the pivot is a light $\frac{1}{4}$-inch brass tube *a* having at its upper end a pointer *c*. Being rigidly connected

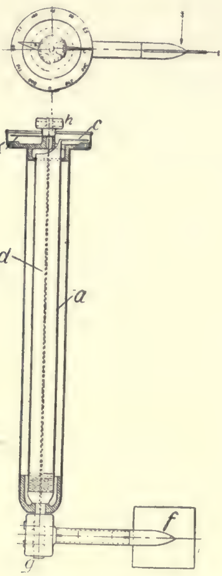


FIG. 40.—Direct reading type current-meter.

to the vane the pointer *c* indicates the exact position of the vane *f*. Now to bring the vane back when acted on by the flow to its position at right angles to the current and the pointer *c* to 0, the thumb screw, *h*, is turned. This thumb screw is attached to the torsion wire *d* and the wire is attached to the pivot *g*. Therefore by turning the screw *h* the vane is moved back and the pointer *k* attached to the thumb screw, to some position, say 175. The torsion in the wire is directly proportional to the pressure on the vane, or the velocity of the water, so the reading will be 175 feet per minute. On the dial the scale, made

from actual tests, is placed, and gives the velocity in feet per minute (see plan view). When a velocity is wanted all that is necessary is to stick the meter in the water with the vane about perpendicular to the current and then rotate the thumb screw till *C* comes to 0. By taking the highest reading of the pointer *k*, *C* being kept at 0, it will be known that the vane is perpendicular to the current.

In this instrument there is no rotation of delicate parts, the vane only moving through an arc of some 30°. Therefore grit and grass have little effect on it. Depending on the torsion of a steel wire for its accuracy it is at all times ready for use and readings can be made as easily as time can be taken from a watch. The vanes are detachable and a set of four goes with each instrument so that the greatest sensitiveness of the instrument may be used for all currents. Thus for a very slow current a large vane is used and for a swift current a small vane. When using any but the standard vane the readings have to be multiplied by a constant. This meter is made jointed so that it may easily be carried in a suit case, its weight being but two or three pounds.

Extensive Measurements.

If the measurements are to be made on a stream of great width and to continue during times of flood, a cable way is suspended over the selected spot and a car is used. The cable is marked every five or ten feet depending on the roughness of the river bottom, and the measurements made at these points every time.

In taking accurate measurements it is well to take a mean of three general methods. These methods are as follows:

Six tenths single point method; numerous experiments have shown that the average velocity of the entire area is found at a depth equal to six tenths the whole depth. Therefore in taking these measurements the meter is held at that depth under the water at each five or ten foot mark on the cable. The average of these mean values gives the average velocity of the section. Surface single point method; here the meter is held but a foot under the water at each point across the stream and the average taken. This average is then multiplied by .85 to .95 to get the average velocity for the section. .95 would be

used for streams having smooth bottoms. The rougher the bottom, the lower would be the average velocity. Integrating method; in this case the meter is lowered slowly and with a uniform motion from the surface to the bottom and back again.

From the gauging car measurements of depth are also taken at each interval. The widths are not taken as the profile made at the start shows the location of each interval.

CHAPTER III.

RECONNOISSANCE OF WATER POWER.

Up to a short time ago the common practice has been to purchase a water power and install water wheels as required by the increasing business, without first obtaining exact knowledge of the true value of the full power of the stream. Now, however, when organized capital and business foresight are the governing factors in nearly all power developments, and each horse power obtained is becoming more and more valuable, it behooves us to know as nearly as possible the actual power to be had under the given conditions. It is seldom indeed that the power to be derived from a stream is not over-estimated. This is usually due to the anxiety of the owner to impress the engineer with the fact that he expects a certain amount of power, and the willingness of the engineer to make his report agree with the owner's wishes.

Any man of average intelligence should be able to make measurements of his water power which will serve as a check on those taken by the engineer, and in most cases can be used as a basis for the preliminary estimates.

Power Measurement.

Water power is measured by two quantities: pounds of water flowing down the stream each minute, and the " fall " or " head." The amount of head should be found by a competent surveyor, and as the limiting factor is the cost of the overflowed lands, the owner should accompany the surveyor when the levels are run to see that no little brooks or drainage ditches are overlooked. Sometimes a very innocent ditch is a drain for many acres of valuable farm land and if you back it full of water you are liable to heavy damages. The surveyor can from time to time set his level up so that the glass is on a level with the crest line of the proposed dam, and by sweeping around over the river

bottoms, get a very good idea of the overflow. A county map is a great aid in getting the acreage of the submerged land.

Having obtained the levels, the area of the reservoir as it will be when full should be roughly approximated, as its extent will be of use in determining the value of the power.

The head in feet being obtained, the next item should be the pounds of water flowing in the stream each minute. Great care should be exercised in determining this item.

To be safe these measurements should extend over several years as the yearly flow varies between wide limits, but usually this is impossible. Measurements taken at any other than during the time of low water will be untrustworthy as it is with the minimum we usually have to deal. In determining these periods of low water, if the observer is a stranger in the vicinity of the proposed dam, he may ask several of the old fishermen and hunters, but should not depend upon merchants and bankers for this most important information. On many of the rivers government reports may be obtained, but the author would advise caution in their use.

The methods of determining the flow are fully explained in Chapter II.

Value of Government Reports to the Hydraulic Engineer.

Wishing to make all possible use of the various reports on rainfall, run-off, etc., annually issued by the United States Government, the writer made an extended study of the subject with the following results:

No reliable data can be obtained from the reports on precipation which will aid in estimating the maximum flood flow. Nor will the rainfall data be of use in predicting the day, week or month when these extreme floods will occur. An average can be taken, but outside of this average are such widely scattered variables that for practical purposes there is no information given.

For example, take the Table XI and note that during August, 1903, the rainfall was 6.93 inches, and in October it was 6.26. Yet there is no corresponding increase in the run-off. Note that for March, 1904, the time of the terrible flood which cost many lives and millions of money, the rain was only 3.08 inches. This flood was caused entirely by a sudden thaw which melted

TABLE XI.

RAINFALL AND RUN-OFF DATA, SUSQUEHANNA RIVER. AVERAGE OF NINETEEN STATIONS.

Month.	1903 Run-Off.	1903 Rain-fall.	1904 Run-Off.	1904 Rain-fall.	Month.	1903 Run-Off.	1903 Rain-fall.	1904 Run-Off.	1904 Rain-fall.
	Cu. ft. per sec. per sq. mile.	inches	Cu. ft per sec. per sq. mile.	inches		Cu. ft. per sec. per sq. mile.	inches	Cu. ft. per sec. per sq. mile.	inches
January....	1.626	2.60	1.280	3.46	July	1.331	4.23	0.800	5.16
February...	3.552	2.50	1.620	2.24	August. ...	1.053	6.93	0.519	4.21
March......	5.023	4.93	4.280	3.08	September .	1.277	1.64	0.413	3.56
April.......	2.910	2.01	2.930	2.79	October.....	1.822	6.26	0.698	3.01
May.	0.628	0.85	1.750	3.64	November..	1.151	2.24	0.498	1.17
June.......	1.115	6.70	1.290	2.99	December...	0.737	2.20	0.407	2.18

Total rainfall for year = 25 inches (1903), and 18.7 inches (1904).

TABLE XII.

RUN-OFF AND RAINFALL DATA FOR VARIOUS RIVERS.

	Date	Drainage area sq. miles	Run-off cu. ft. per sec. per sq. mile
Chippewa R., Eau Claire, Wis.	1904	6,740	1.362
Flambeau R,. Ladysmith, Wis.	1904	2,120	1.230
Wisconsin R., Merrill, Wis.	1904	2,630	1.850
Rock R., Rockton, Ill.	1904	6,150	*.761
Illinois R., Minooka, Ill.	1904	6,480	1.953
Youghiogheny R., Friendsville, Md.	1904	295	†2.120
Mahoning R., Youngstown, O.	1904	958	1.247
Licking R., Pleasant Valley, O.	1904	696	.792
New R., Radford, Va.	1904	2,725	.968
New R., Fayette, W. Va.	1904	6,200	.929
Greenbrier R., Alderson, W. Va.	1903	1,344	1.480
Greenbrier R., Alderson, W. Va.	1904	1,344	.911
Scioto R., Columbus, O.	1904	1,051	1.017
Olentangy R., near Columbus, O.	1904	520	1.103
Wabash R., Logansport, Ind.	1904	3,163	1.600
Tippecanoe R., Delphi, Ind.	1904	1,890	off 10%
White R. (E. Branch), Shoals, Ind.	1904	4,900	.947
French Broad R., Oldtown, Tenn.	1904	1,737	1.050
Tennessee R., Knoxville, Tenn.	1904	8,990	.842
Pigeon R., Newport, Tenn.	1904	655	1.070
Nolichucky R., Granville, Tenn.	1904	1,099	1.030
Halston R. (S. Fork), Bluff City, Tenn.	1904	828	.862
Watauga R., Elizabethton, Tenn.	1904	408	1.280
Little Tennessee R., Judson, N. C.	1904	675	1.650
Tuckasegee R., Bryson, N. C.	1904	662	1.470
Hiwassee R., Murphy, N. C.	1904	410	1.290
Hiwassee R., Reliance, Tenn.	1904	1,180	1.200
Nottely R., Ranger, N. C.	1904	272	1.140
Susquehanna R., McCall's Ferry, Pa.		1,370	
Average value.		1.23	1.357

* Minimum is 62% too low.
† Maximum is 58% too high.

TABLE XII.—(*Continued.*)

Name of River.	Date.	Drainage area sq. miles.	Run-off cu. ft. per sec. per sq. mile.
Colorado R., Yuma, Arizona	1903	225,049	.069
Gila R., Yuma, Arizona	1903	....	.085
Virde R., McDowell, Arizona	1903	6,000	.0533
Salt R., McDowell, Arizona	1903	6,260	.041
Salt River, Roosevelt, Arizona	1903	5,756	.061
Tonto Creek, Roosevelt, Arizona	1903	1,030	.038
Grand R., Elmwood Springs, Colo	1903	5,838	.47
Whiterocks R., Whiterocks, Utah	1903	114	1.23
Bear R., Collinston, Utah	1903	6,000	.20
Sevier R., Gunnison, Utah	1903	3,986	.027
San Pitch R., Gunnison, Utah	1903	836	.045
Humboldt R., Oreana, Nev	1903	13,800	.009
Humboldt R., Golconda, Nev	1903	10,780	.0158
Humboldt R., Palisade, Nev	1903	5,014	.066
South Fork of Humboldt R., Elko, Nev	1903	1,150	.153
East Fork of Walker R., Yerington, Nev	1903	1,130	.103
West Fork of Walker R., Coleville, Cal	1903	306	1.02
Walker R., Wabuska, Cal	1903	2,420	.0704
Carson R., Empire, Nev	1903	988	.43
East Fork of Carson R., Gardnerville, Nev	1903	381	1.21
West Fork of Carson R., Woodfords, Cal	1903	70	1.70
Truckee R., Tahoe, Cal	1903	519	.40
Truckee R., Vista, Nev	1903	1,519	.52
Truckee R., Pyramid Lake, Nev	1903	2,130	.40
Truckee R., Mystic, Cal	1903	955	.79
Donner Creek, Trucker, Cal	1903	30	2.57
Cache Creek, Yolo, Cal	1903	1,280	.43
Cache Creek, Lower Lake, Cal	1903	500	.67
Feather R., Oroville, Cal	1903	3,350	2.11
Stony Creek, Fruto, Cal	1903	760	.71
Sacramento R., Red Bluff, Cal	1903	9,295	1.50
Tuolumin R., Lagrange, Cal	1903	1,501	1.82
Merced R., Merced Falls, Cal	1903	1,090	1.28
King R., Sanger, Cal	1903	1,742	1.31
Yule R., Portersville, Cal	1903	437	.34
Kern R., Bakersfield, Cal	1903	2,345	.32
Santa Anna R., Waumpingo, Cal	1903	182	.46
Mohave R., Victorville, Cal	1903	400	.37
Yakima R., Kiona, Wash	1903	5,230	1.12
Yakima R., Union Gap, Wash	1903	3,300	1.83
Naches R., North Yakima, Wash	1903	1,000	2.57
Triton R., North Yakima, Wash	1903	289	3.15
Missoula R., Missoula, Mont	1903	5,960	.571
Brittmort R., Grantsdale, Mont	1903	1,550	1.007
Weiser R., Weiser, Idaho	1903	1,670	.80
Boise R., Boise, Idaho	1903	2,450	1.28

the accumulated snow, and such a thaw may occur at any time of the winter.

In the summer we are no more sure of the flood periods. There may have been a drought and then a terrible rainfall. In this case the dry soil takes up so much of the precipation that there is no flood. Another year is damp but has slight precipation. A heavy rain comes and the soil being saturated, all the rain runs into the river and a flood is the result. Therefore it is impossible to predict the flood periods from rain fall data. For the same river the run-off changes between wide limits, for the same rainfall and for the same time of the year. The rainfall at one station bears no resemblance to that at other stations on the same drainage area.

From the government reports on run-off per square mile of drainage area, data can be obtained, which is of use in computing the yearly power to be depended upon. If the reports have extended over a period of several years, a table like table XI, covering that period will give an annual run-off per square mile which will be very close to the actual. An enterprise properly handled will not figure on the results of the poorest year, but will base the investment on the average income for a long period. In this case the above average run-off for all the years will furnish a safe basis for estimating the power.

As will be seen by referring to Tables XI to XIII, the run-off per square mile of drainage area, varies somewhat for different rivers, and for the same river it varies with the year, but the average results are not so erratic as to preclude their use as a basis for estimates.

It will be noticed that the Western rivers are much more erratic than those of the Eastern or Middle States, but even in their case, by using judgment in considering the location of those rivers having abnormal run-off, a safe average value can be estimated for the average annual run-off per square mile of drainage area.

The *maximum flood* to be expected, no matter when it occurs, is a matter of great importance, as the safety of the enterprise largely depends upon it.

There is one way, and only one way, to compute this, whether done by the Government or the engineer, and that is by measurements made on the spot. Where the Government gauging

TABLE XIII.

INCHES OF RAINFALL, SUSQUEHANNA RIVER, STATIONS 1 TO 19, 1904.

Station No.	January	February	March	April	May	June	July	August	September	October	November	December
1	3.21	2.18	3.27	2.47	1.10	3.61	3.27	4.20	3.86	4.16	1.26	2.62
2	4.29	3.00	3.06	2.84	2.40	4.00	4.74	4.55	4.08	3.49	1.18	2.49
3	5.39	3.24	2.68	3.80	2.49	2.35	8.85	4.79	3.28	3.06	1.11	3.88
4	3.73	1.75	2.98	2.59	2.62	4.60	5.92	4.41	4.51	3.09	1.86	2.08
5	3.57	2.80	5.28	3.59	2.82	2.71	5.20	7.13	4.66	4.45	2.07	2.64
6	2.37	1.67	2.75	1.99	2.19	1.73	4.54	6.33	4.34	4.61	1.98	1.87
7	4.63	2.85	3.72	3.09	3.06	1.22	5.98	4.49	5.25	3.06	1.50	3.75
8	3.62	2.10	2.85	1.55	4.03	2.57	7.55	4.50	5.02	3.29	.84	2.68
9	2.11	1.16	2.11	2.51	2.66	2.76	4.73	3.12	2.88	3.31	.49	1.12
10	2.70	1.83	2.92	3.54	5.61	2.01	5.48	3.10	2.80	3.82	1.07	1.80
11	3.68	1.77	3.12	3.87	5.31	3.39	4.79	4.85	2.13	2.02	.62	1.87
12	4.56	2.39	3.59	2.99	4.39	4.31	6.35	3.08	3.69	2.79	.98	2.05
13	2.69	1.48	2.47	1.97	4.00	3.68	6.54	4.07	2.74	2.68	2.75	2.72
14	3.45	3.85	3.15	2.81	5.06	2.03	4.20	3.80	3.01	2.46	1.05	2.10
15	2.47	1.56	2.79	2.27	4.44	1.94	4.53	3.76	2.63	1.57	.56	1.13
16	3.18	2.21	2.52	2.77	5.00	4.56	3.80	3.61	3.52	2.01	.57	1.15
17	3.47	1.53	3.67	2.57	4.02	3.33	2.70	3.31	3.38	2.08	.69	1.81
18	....	....	....	....	....	....	....	....	....	....	....	....
19	3.08	3.06	2.60	2.95	4.32	3.04	3.78	2.68	2.30	2.24	.40	1.60
	62.20	40.43	55.53	50.16	65.52	53.84	92.95	75.76	64.10	54.19	20.98	39.36
Average.	3.46	2.24	3.08	2.79	3.64	2.99	5.16	4.21	3.56	3.01	1.17	2.18

station has been established ten years or more, the maximum flood reported is a safe figure. But when there has not been such a report the engineer must visit the site and make the measurements. The *highest flood* is always a matter of history among the inhabitants living near the river, and even if the highest flood did not leave marks along the banks, these natives may be relied upon. A house was moved at this place, a drift log was left at another place, and so on.

From such data the engineer finds the slope of the surface of the flood for a distance of say 1,000 feet each side of the site. A profile is taken at 100 foot intervals. From this data and the formula $Q = A\, C \sqrt{r\, s}$, given in Chapter II, the quantity of water flowing in cubic feet per second is found. C would be taken at say, .05. The slope, s, has been comptued, and r, the mean hydraulic radius.

Where the engineer has the opportunity the surest way to obtain the greatest flood to be expected is to visit the site during a flood and find the slope of the water's surface. Then from the marks showing the stage of the greatest flood, determine the proper level of the surface. The slope will be approximately the same for both floods and r is found from the profiles by dividing the area of the section by the wet perimeter. A, the area of the average section, is also found from the profiles.

Frizell gives the formula $Q = 17.35 \sqrt{\frac{8006}{A}}$, where Q is the maximum flood that may ever be expected, per square mile of drainage area, and A, is the number of square miles of the drainage area.

Applying this to the Susquehanna river, $Q = 17.35 \sqrt{\frac{8006}{27666}} = 9.54$. $27{,}666 \times 9.54 = 262{,}933$ cubic feet per second. The actual flow of the Susquehanna on March 8, 1904, was 700,000 cubic feet per second. It will be seen therefore that Mr. Frizell's formula is not reliable in estimating the greatest flow to be expected.

Relation of Pondage and Reservoirs to the Valuation of Power.

For judging a power it is of prime importance to know the use to which it is to be put, *i.e.*, one should have a general idea of the load curve.

For example, consider a small stream having an available head of 20 feet and a flow of 5000 cubic feet per minute and a storage area of 200 acres. First, assume that it is desired to drive a paper mill requiring 300 h.p. This stream, however, can not produce more than 190 theoretical h.p., and since a paper mill runs 24 hours, the power is too small.

Next suppose that the power is to be used to drive a factory, which runs but 10 hours per day. One acre of water one foot deep contains 43,560 cubic feet, and weighs 2,722,500 pounds; therefore the storage capacity is $43{,}560 \times 200 = 8{,}712{,}000$ cubic feet. The flow for 14 hours during which the plant is idle would fill $\dfrac{5000 \text{ cubic feet} \times 14 \text{ hours} \times 60 \text{ minutes}}{43{,}560 \text{ cubic feet per acre}} = 96.4$ acres, so it is seen that 200 acres is amply large for this installation and the total energy of the stream can be utilized. The total energy is $\dfrac{5000 \text{ cubic feet} \times 62.5 \text{ pounds, } 20 \text{ feet, } 24 \text{ hours}}{33\,000} = 4550$ h.p. hours. which will give $\dfrac{4550}{10} = 455$ h.p. for 10 hours.

Assuming that it costs $25.00 per horse power year to produce the power from coal, the water power is worth $11,550 per year. In this case it is purely a question of finance, whether the stream is to be utilized or not.

Lastly, suppose that the power can be used to light several small towns near by and a city of 30,000 inhabitants six miles away. The average period during which the lamps are on covers about four hours so that the reservoir must store during 20 hours and have a capacity equal to

$$\frac{5000 \times 20 \times 60}{43560} = 137.8 \text{ acres,}$$

therefore it is seen that 200 acres will be amply sufficient to store the water so that the entire energy of the stream can be utilized during four hours and the power will be $\dfrac{4550}{4} = 1137.5$ h.p. or $1137.5 \times .0746 = 848.5$ kw. (theoretical). The total available energy is $848.5 \times 4 = 3392$ kw.-hrs., which at five cents per kw.-hr. is worth $61,612.00 per year.

From the above it will be seen how important it is to fit the power to the market or the market to the power. The con-

stantly grinding knives of the paper mill demanded a strong, constant flow, the factory an average ten hour load with good storage capacity. The third case shows the most favorable condition for a large income and should make it apparent that it is worth a large initial investment in transmission lines to so dispose of the power.

Suppose now there is no reservoir and that all the water that comes down the stream is being used. A sudden peak comes in the load and as a result the water is drawn from below the crest of the dam and, since the entire flow of the stream is being used, the pond will not fill up again and the next peak will cause it to be drawn still lower, with the result that the plant is soon compelled to shut down. In other words, if there is no reservoir, the value of the power is determined by the minimum flow per minute, while in the last two cases if there is ample reservoir the value of the power is more than doubled, as not only can the "peaks" be taken care of, but power can be stored during the idle hours.

The amount of evaporation on the surface exposed to the sun is proportional to the area and therefore is increased when a reservoir is formed. The evaporation is in most cases a negligible quantity, though under certain conditions it should be taken into account.

In the above we have in every case taken the theoretical power. The actual power delivered to the customer will vary from 50 to 70 per cent. of the theoretical.

By referring to Table LXXV, the value of a reservoir for any head can be estimated. Referring to this table, an acre under a 20-foot head will give $\frac{27.50}{4} = 6.88$ h.p. for four hours and $\frac{27.50}{8} = 3.44$ h.p. for eight hours. Knowing the value of the land and the power, it can be determined how much land to buy for reservoir purposes. In planning the reservoir due allowance must be made for the climate at that particular place. If it is in the north where the ice freezes several feet thick, the reservoir must be deep enough to allow for the ice. It should never be less than two feet. The ice, besides reducing the storage capacity, reduces the head by an amount equal to three-fourths the thickness of the ice.

The reservoir may be drawn down two feet, in which case its area would have to be but little more than half as great. In any case, the amount the pond is to be drawn down should be determined at the start and the plant designed accordingly. Of course, the greater the head, the more can the pond be drawn down without seriously affecting the regulation of the plant, but in *no case should the water be drawn down so low that it will not regain the level of the dam's crest in time for the next run.*

In the foregoing we have only treated of storing enough water to carry the power for a day or so, but it frequently happens that sufficient pondage can be secured to supply the deficient water through the four or five months of low power.

In Fig. 41 is given a set of curves showing the horse-power and flow of a river for the years 1899–1905, these measurements were taken on a weir every day by a competent man. They are interesting in many ways. Suppose now that it is desired to provide a large reservoir so as to take care of such a year as that in 1901, when for 160 days the power averaged only 275 h.p., and render 1000 h.p. available at all times.

Fig. 42 shows the flow, in thousands of cubic feet per minute, covering the same period. By comparing the flow curves with the power curves one can trace the variatons of head and its effect on the power.

Twenty miles up the river there is a valley where, by building a dam, a reservoir 12 miles long with an average breadth of 1000 feet and depth of 10 feet can be created. The head at the power house is 70 feet, therefore this reservoir will supply

$$\frac{5280 \text{ ft.} \times 12 \text{ mi} \times 1000 \text{ ft.} \times 10 \text{ ft.} \times 62.5 \text{ lbs.} \times 70 \text{ ft.}}{160 \text{ da.} \times 600 \text{ min.} \times 33000} = 874 \text{ h.p.}$$

for 160 ten-hour days.

A gate is placed in the reservoir by which the amount of water is regulated. This gate may be operated by electricity from the power house. The pond above the *power* dam is kept at all times just full to the crest of the dam.

This upper reservoir has nothing to do with the head at the lower dam as there is 20 miles between the two dams, its only office being to catch the flood water and hold it for the future dry season. It will also incidentally aid in preventing heavy floods if it has been well drawn down during the dry weather and before the spring floods come on.

In storing water for long periods the evaporation must be taken into account. This, of course, varies between wide limits, being greater for dry warm climates than for cooler and more elevated localities. About 1/16 inch per day is a safe

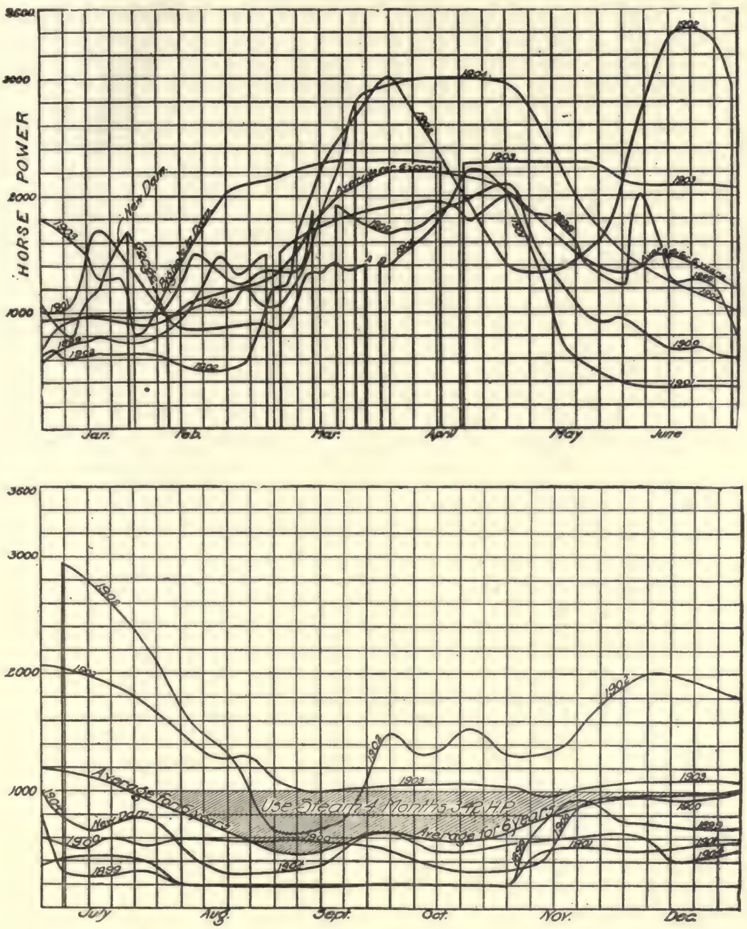


FIG. 41.—Curves showing horse power in river.

figure for the evaporation during the 12 months. This in the above example would make an evaporation of about 14 inches of the water in the reservoir.

The value of such a reservoir depends on the price for which

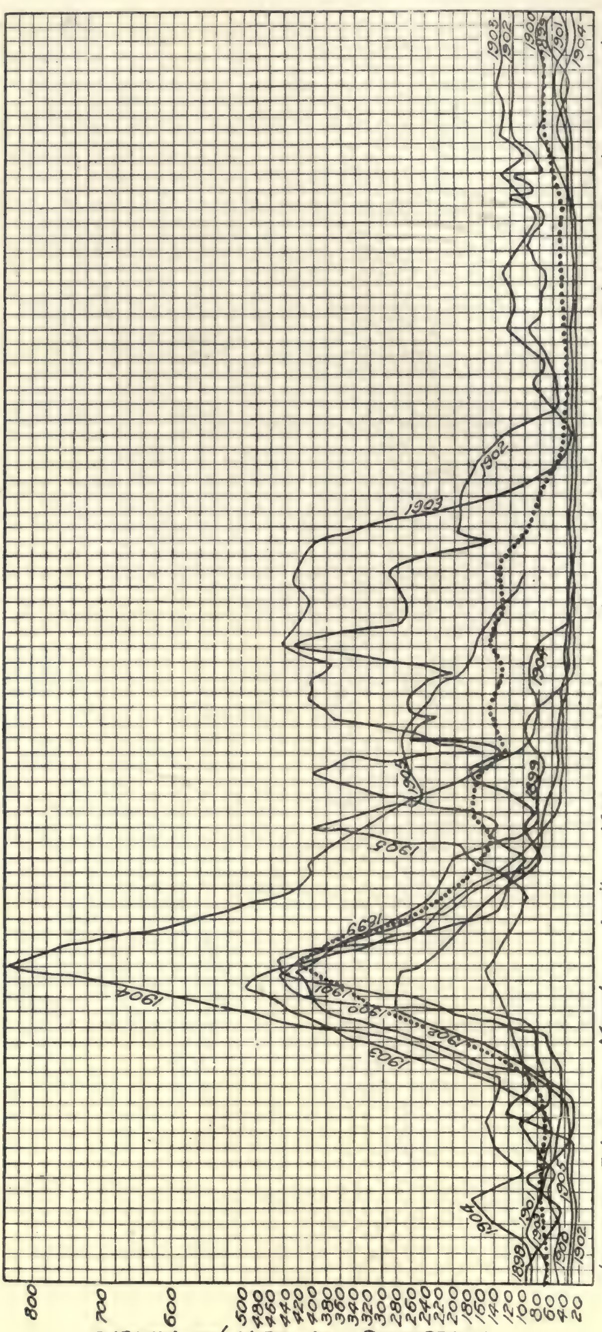


FIG. 42.—Curves showing flow in river.

the power may be sold and the cost of a steam plant to develop the same power, the annual cost of operation being taken into consideration. In comparing the value of the large reservoirs and an auxiliary steam or gas engine plant, it must be borne in mind that with the reservoir peak loads can be taken care of. In the above example the reservoir is worth fully 1600 steam h.p., depending on the size of the peak.

A steam plant of 1600 h.p. would cost all erected, about $80,000, and the yearly 160 days of running, during low water, and cost of operation, would be $21,000 including interest, depreciation, attendance, coal, etc. The cost of operation of the reservoir would only be the interest and depreciation so that the added cost of operation of the steam plant would pay interest on a very large expenditure for reservoirs.

Frequently several smaller reservoirs may be found giving in the aggregate the desired capacity. It is a good plan where there are several power owners along the river, to all combine in building reservoirs as all will be equally benefited.

Where several water powers are situated on the same river and all using the power for about the same purposes, each plant gets the use of all the reservoirs above it, as each plant is drawing on its own pond and all the rest above, thus passing all the water it can use to the next user down the stream.

Penstock with Reservoir.

The possession of a reservoir in connection with a penstock can, under favorable conditions, increase the capacity of the penstock two fold or more. Or, with a reservoir the penstock need have but half the capacity that it would have to have without it.

If the penstock supplies water for a power operating 24 hours per day the capacity of the reservoir need only be such as will take care of the peak loads, but where it operates, say 10 hours per day it will have to store the flow through the penstock for 14 hours. Without this reservoir at the power house end, the penstock would be out of use when the factory was shut down.

It will therefore be seen that it is of the utmost importance to provide a reservoir at the power house end where long expensive penstocks are used and where the load is fluctuating or of short duration.

A splendid example of a penstock and reservoir is that of the Mill Creek Power Plant near Redlands, Cal., and a short study of this plant should prove instructive.

Mill Creek is a tiny mountain creek which one could easily jump across or wade and hardly wet the ankles. All the water

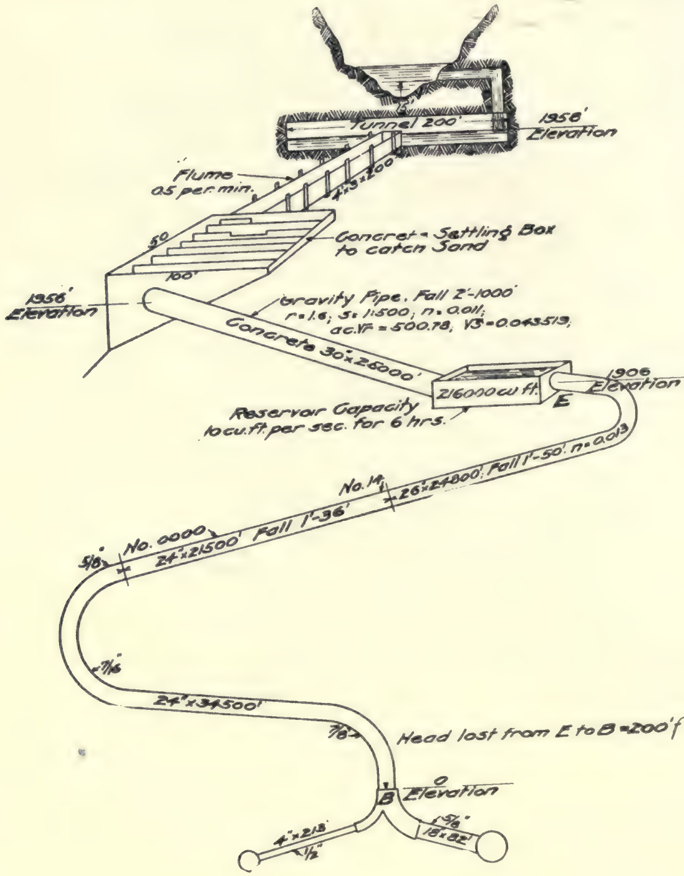


FIG. 43.

that can be safely relied on is 20 cubic feet per second, and to get even this small amount a tunnel 200 feet long had to be run under the bed of the stream to collect every drop of seepage. Fig. 43 is a sketch showing the system.

The tunnel in collecting the flow and seepage from the creek

picks up a good deal of sand. This sand is separated from the water by means of a settling tank as shown. A rectangular, open timber penstock carries the water from the tunnel to this settling tank. The water entering the tank flows from compartment to compartment over the partitions and leaves the sand to settle in the quiet water. There are seven partitions all of which, except the middle one, are three feet below the surface of the water. From the basin the 20 cubic feet per second passes into a 30 inch gravity penstock having a fall of one foot in 500. This pipe is called a gravity pipe because it follows the hydraulic gradient, there being but 4 inches of pressure head on it at any point. Its fall is regular, a perfect grade being necessary. This part of the pipe line was built of concrete in the proportion of one of cement to three gravel and in sections 24 inches long. The thickness of shell is $2\frac{3}{4}$ inches. Fig. 44 shows how two sections are joined together by means of the

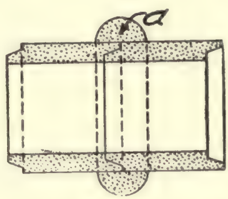


Fig. 44.

concrete ring *a*. The sections were built in camps along the line and cost $1 per foot to make. To lay the pipe including the digging of the trench three feet deep cost $1 per foot. Conditions were the worst possible for cheap results.

The gravity penstock empties into a reservoir formed by a dam across a ravine and has a capacity sufficient to furnish ten cubic feet per second for six hours. As will be shown later the loss of head between the reservoir and the power house is about 376 feet leaving an effective head of $1906 - 376 = 1530$ feet.

The reservoir will store $10 \times 60 \times 6 \times 10$ cubic feet or 216,000 cubic feet.

Ten cubic feet per second used under a head of 1530 feet would give 1550 theoretical h.p. Can there be a stronger argument for a reservoir than this? Suppose 60 per cent. of this is delivered at $50 per h.p., the annual income from a reservoir costing perhaps $5000 would be $46,500.

Suppose that there is no reservoir, (Fig. 43) and that the pressure pipe connects directly with the gravity line. Then if the peltons were all suddenly started, the water in the pressure pipe would get up a high velocity before that in the gravity pipe got started. The result would be a vacuum in the gravity pipe. Therefore this part of the pipe would have to be calculated to withstand a vacuum or relief valves would have to be provided.

The pressure which will collapse a steel pipe is found from

$$P = 806{,}000 \frac{t^2}{l\,d}$$

where t = thickness of metal, l = length in inches between flanges or joints and d = the diameter of pipe in inches.

From the reservoir to the power house the fall is 1906 feet. and to conduct the water a steel pipe is used. Here the question of how much water the pipe should carry comes up. If there were no reservoir the answer would be 20 cubic feet per second, but having a reservoir, a pressure pipe having a sufficient capacity to take care of the peak load must be provided. This of course depends on the purpose for which the power is used, but it is very seldom indeed that the peak does not exceed twice the average load. On this basis the pipe should deliver 40 cubic feet per second. (See Chapter II.)

Now let us take up in detail the design of this complicated penstock, and calculate the head lost in each division.

(1) Rectangular open wooden penstock 200 feet long, three feet deep and four feet wide. $r = 1.2$. $\sqrt{s} = ?$ $Q = 20$ cubic feet per second.

The velocity v is found by dividing the quantity Q by the area A and $= 1.666$ feet per second. $v = C\sqrt{r} \times \sqrt{s}$. $n = .01$. From Table I, $C = 161.5$. Substituting in formula for v, $1.666 = 161.5 \times 1.118 \times \sqrt{s}$ and $\sqrt{s} = .0091$. Referring to table V the slope corresponding to one foot in 110 feet or say the fall in the 200 feet should be two feet.

(2) 30 inch concrete gravity penstock 25,000 feet long.

$$Q = 20. \quad n = .011. \quad Q = A\,C\sqrt{r}\sqrt{s}.$$

From Table III, $A\,C\sqrt{r} = 500.78$. Substituting, $20 = 500.78 \times \sqrt{s}$ and $\sqrt{s} = .0399$. From Table V we find the correspond-

ing slope to be one foot in 590. The slope actually adopted at Mill Creek was one foot in 500, so it is evident that the coefficient assumed was .01. This, in the opinion of the writer, is working too close for good practice. Certainly a concrete pipe having joints every two feet is more rough than good planed plank.

(3) 26 inch steel pipe 2480 feet long, riveted.

From Table III we find that for a 26 inch clean cast iron pipe $A\ C\sqrt{r} = 302.90$. We will assume that a riveted steel pipe has the same rugosity. As above explained the pipe should. deliver at least 40 cubic feet per second. Then $Q = A\ C\sqrt{r} \times \sqrt{s}$, or, $40 = 302.9 \times \sqrt{s}$ and $\sqrt{s} = .132$. From Table VI we find the slope = one foot in 55. $v = 10.8$ feet per second.

(4) 24 inch steel riveted pipe 2150 + 3450 feet long.

From Table III $A\ C\sqrt{r}$ for cast iron pipes = 247.57. $Q = A\ C\sqrt{r} \times \sqrt{s}$ and $40 = 247.57\sqrt{s}$, $\sqrt{s} = .00161$. From Table V the slope = one foot in 36 feet.

In this way the total head lost between the reservoir and power house is found to be about 200 feet. Thus, if one foot head is lost for every 36 feet of the 24 inch pipe, the length of the pipe divided by 36 gives the total loss of 155 feet. Adding this to the loss in the 26 inch pipe a loss of nearly 200 feet is obtained.

The gravity pipe line pierced the mountains in 19 places in order that its center line might coincide with the hydraulic gradient, The total length of tunnels was 7,490 feet. This should encourage the engineer who has the ordinary proposition under consideration.

Ice Evils.

In making a reconnoissance, the engineer should always take into consideration the probable trouble to be expected from cold weather.

The author recently (1906) made a tour of the great water power plants in the northern states during the extremely cold season, and made a special study of the ice troubles experienced in operation. A resumé of the conclusions drawn from the investigation is given below:

Narrow channels parallel with the dams and head works were seldom troubled with ice; plants having short mill ponds ending in rapids or long shallow ponds ending in rapids were invariably troubled with anchor ice; in ponds which were frozen over to a good depth the anchor ice lost its form before it got to the racks; tail races which were not protected were frozen over and clogged unless they were deep enough to take care of the water; turbines running in unprotected steel flumes were, in some cases, so bothered by water freezing in the flumes that housings had to be built around the flume and fires kept burning;

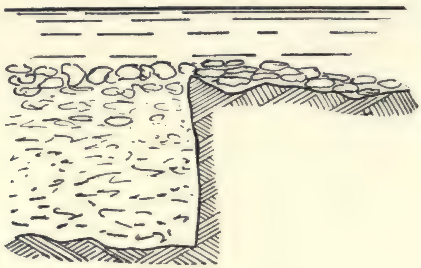


FIG. 45.

the lowest flow in the winter was about the same as that during the low water period in summer.

SOUNDINGS.

Test Holes.—The natural impulse of engineers and investors is to rush into a job without preliminary soundings which are to tell what the foundations will be. Not long ago the author was called upon to inspect a site for a dam and found that because the banks on either side of the river were stone, the owners had jumped at the conclusion that the bottom was also stone. $20,000 was spent in getting ready to build a masonry dam and then it was found that there was no rock bottom. Often the rock bottom drops perpendicularly down for from 50 to 100 feet as shown in Fig. 45. The continual wearing of the valleys fills the river bed below the ledge with sand and gravel, and to the eye presents the appearance of a uniform river bed. Unfortunately this condition is most likely to exist

at the narrowest part of the river and between cliffs, just where the dam would naturally be wanted.

Again there may be mud holes under what is apparently solid rock bottom, as at *M*, Fig. 46, and there may be several inches of mud between the layers of rock.

Another condition often found is shown in Fig. 47. A foundation is excavated for the power house in the hard blue clay and unless soundings are made it would not be suspected that at the shore side the foundations rested on only a few inches of clay, the rest being quicksand. In this case the power house would settle badly and in time be destroyed.

FIG. 46.

These three cases (there are many more) should serve to urge the engineer and investor to spend some money on sounding the bottom.

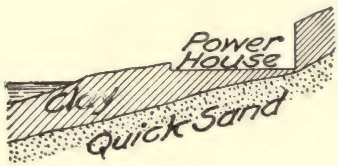


FIG. 47.

Rock.—On extremely large and important work a diamond drill is used in getting a sample of the river bed. The diamond drill brings to the surface a solid core of rock, an examination of which tells exactly the nature of the bottom. An accurate measurement must be made of the depth of the diamond below the surface so that if there are seams in the rock their thickness may be determined.

Soft Bottoms.—The common way of sounding soft bottoms is to drive a gas pipe of from $2\frac{1}{2}$ to 4 inches diameter in the same

way a drive-well is sunk. The pipe is divided into 8 foot sections and driven with a plug in the top end. Every 2 feet, 4 feet, or 8 feet, the plug is removed and the pipe cleaned out with a sand pump. Water is used to loosen up the materials in the pipe. By examining the materials thus removed a record is made of the bottom. It is a good plan to empty the pump each time into a tall glass jar, the water being drained off. The exact composition of the materials may then be ascertained. From time to time the pipe will become plugged with stone, in which case a drill such as shown in Fig. 115 is screwed into a gas pipe and two men, using it as a churn drill, drill out the obstruction.

The driver used is usually made in the form of a small pile driver, a heavy section of some hard-wood tree being used for the hammer. The hammer may be lifted by man power. Where only a few holes are required a 2 inch pipe may be driven with a heavy post maul.

Much may be learned of the character of the substratas by sounding with a ⅝-inch round iron rod, and the engineer should never be without such a rod when making a hasty preliminary inspection. Only a year ago the author learned this at a cost of about $6000. After the contract was secured, a little ⅝-inch rod showed that where it was without a doubt solid granite, the bottom dropped down 14 feet, making much more excavation necessary than had been figured on, and necessitating the building of more dam.

By listening to the rod you can tell whether any rock struck is solid or only a boulder.

For most soft soils a common 2-inch wood auger having a auger handle, or, for deep boring, a handle several feet long, is handy for sounding. There are several earth augers on the market, but for soft loams and clay a common auger is good.

In this manner one or two men can sound over a large area.

Flowage Height.

When a dam is to be built it becomes a question of great moment, just how high the water will be raised at different points above the dam. There may be a city up the river whose drainage rights must not be affected, or there may be another water power above the proposed dam whose tail water limits

the height of the proposed dam. In these cases it must be known what the flowage height will be.

Water flowing in the stream obeys the same laws as when flowing in a canal or penstock. It merely becomes a greater question of judgment in selecting the coefficient of roughness.

The first step is to go over the reservoir with a competent surveyor and take accurate profiles of the cross-section similar to Fig. 48, say at half-mile intervals. Here $A\,B$ is the assumed elevation of the water when backed up by the dam, $C\,D$ is a line on the exact level of the dam's crest, $E\,F$ is the level of the water in the river at the particular section before the dam is built. While on the ground decide about what the coefficient of roughness will be, making allowance for bends in the river, undulating ground, stumpages, etc.

FIG. 48.

The fall F in feet between any two points along the stream above the dam can be found from the formula

$$F = \frac{v^2\,D}{C^2\,r}$$

wherein F is the fall between two consecutive points where the section of the river has been determined, v is the velocity of the stream in feet per second, D is the distance in feet between the points, C, a coefficient depending upon the ratio of the wet perimeter to the cross-section of the stream and r is the mean hydraulic radius.

$$v = \frac{Q}{A}$$

wherein Q is the cubic feet of water flowing per second, and A is the area of the cross-section in square feet.

C may be taken from the table given below.

$$r = \frac{A}{P}$$

wherein P is the length of the wet perimeter.

The area $A = A\ B\ F\ G\ E.$ can be estimated from the profile curves of the cross-section at the point in question.

Taking the profiles of the cross-sections every half mile and solving for F in the above formula a curve of flowage height, F can be plotted.

EXAMPLE.—A river having a flow of 100,000 cubic feet per minute is to be dammed with a 12-foot dam. At one mile intervals the sections 1, 2, 3, etc., are taken each at the middle of the interval.

Fig. 49 shows the section one-half mile above the dam. For

TABLE XIV.

VALUES FOR C COMMONLY USED BY ENGINEERS.

For rivers, and canals in earth. Fairly regular.

For values of r less than 0.5	C = 30
" " " " from .5 to 1	C = 45
" " " " " 1 " 2	C = 55
" " " " " 2 " 3	C = 65
" " " " " 3 " 4	C = 80
" " " " " 4 "10	C = 100
" " " " " 30 "75	C = 125

the first mile there should not be more than $\frac{1}{8}$-inch fall, so $A\ B$ will be assumed to be on the same level as the top of the dam $C\ D$. With a planimeter, or otherwise, the area $A = 4800$ square feet may be found. The wet perimeter $P = 480$ feet, therefore $\frac{4800}{480}$ = hydraulic mean radius = $r = 10$. From Table XIV C would be 100.

$$v = \frac{\text{cu. ft. per min.}}{A \times 60}$$

$$v = \frac{100{,}000}{4800 \times 60} = .0347. \quad D = 5280.$$

$$F = \frac{(.0347)^2 \times 5280}{(100)^2 \times 10} = .00006 \text{ ft.}$$

Profile 2 (Fig. 50) is located a mile above the first section and $1\frac{1}{2}$ miles above the dam. Profiting by the first calculations $A\ B$ is placed one inch above $C\ D$ (Fig. 48) the area A then $=$ 2600 square feet and $r = 3.24$, $D = 5280$, $v = \dfrac{100{,}000}{2600 \times 60} = .64$.

$$C = 70. \quad F = \frac{(.64)^2 \times 5280}{(70)^2 \times 3.24} = .0065 \text{ feet,}$$

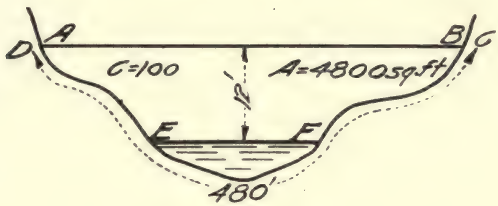


FIG. 49.

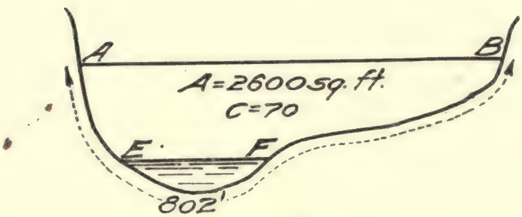


FIG. 50.

and so on with all the sections taken. When a section is reached where the area A divided into the flow Q gives exactly the same velocity as that found in the stream at that section before the dam is built, we know that we are where the dam does not affect the flow at all. In other words the line $A\ B$ coincides with the normal surface of the water in the river at that point before the building of the dam.

COST OF SURVEYS.

The cost of a survey depends entirely on the density of underbrush, and forests, on the variation of levels and on the weather.

Where it is only desired to get an approximate and hasty

level showing the possible head obtainable a level may be run without checking back, for about $2 per mile. More accurate levels such as would be used for the basis of an investment, would cost about $5 per mile. Precise levels will cost $20 to $30 per mile.

A fairly accurate survey of the overflowed land can be made for from $0.25 to $0.50 per acre. This includes running the levels and staking out the high water level.

Topographical surveys with about 50 foot contour intervals will cost, for densely wooded and irregular surface, about $200 per square mile, for more open country about $80 to $90, and for extensive valleys, bare of all brush and trees and with gently sloping sides, about $50.

The same surveys with 5-foot intervals and very precisely made, may cost from $1000 to $2000 per square mile.

Engineer's Report.

GOVERNMENT REPORTS.

Great importance is attached by all capitalists to the reports of the Government. It is the only unbiased report they have to rely upon. It is therefore well to give the Government reports a prominent place. If these reports, as is often the case, give a minimum flow which there is reason to believe is too small or too large, every effort should be made to obtain from the office of Hydrography a detailed description of exactly how the measurements were taken. A visit to the gauging station may be necessary to determine the value of the reports.

All the topographical maps prepared by the department, relating to the drainage area should be procured. The Weather Bureau of the Department of Agriculture should be consulted and the rainfall for the driest year in ten tabulated and the average taken as in Table XIII. It is also a good plan to prepare curves showing the rainfall (average) for each month on the whole drainage area for, say, ten years.

The run-off reports furnish data for another curve showing the average run-off over a long period.

ENGINEER'S MEASUREMENTS.

(1) From the measurements made by the engineer curves are plotted showing the run-off for as long as they were taken.

TABLE XV

RUN-OFF AND RAIN FALL DATA.

Month.	Branch of River to be developed: Rainfall inches. (a)	Run-off cu. ft. per sec. per sq. mile. (b)	Ratio $\frac{b}{a}$ (c)	Run-off of Similar River. cu. ft. per sec. per sq. mile.: Sta Y (d)	Sta Z (e)	$\frac{d+e}{2} = f$ (f)	Ratio. $\frac{f}{b}$ (g)
June......	2.70	2.420	0.896	0.56	0.46	0.510	0.211
July.......	6.75	5.854	0.807	0.44	0.47	0.455	0.078
August....	0.94	0.818	0.870	0.27	0.31	0.290	0.356

TABLE XVI.

RUN-OFF AND RAIN FALL DATA. (DRY YEAR).

	Branch of River to be Developed.		Run-off of River to be Developed.	
	Rainfall dry year. inches.	Run-off. $h \times c = i$	Cu. ft. per sec. per sq. mile. $i \times g = j$	Cu. ft. per sec. per 570 sq. miles. $570 \times j = k$.
Month.	(h)	(i)	(j)	(k)
June......	1.67	1.496	0.315	179.55
July.......	3.88	3.364	0.261	148.77
August....	3.88	3.364	0.261	148.77

TABLE XVII.

RESERVOIR EVAPORATION.

inches.	Cubic feet corresponding to area 15 sq. miles. area $\times l = m$
(l)	(m)
6.10	212,572,800
6.90	240,451,200
5.60	195,148,800

TABLE XVIII.

WATER FOR MINIMUM YEAR

	Total Run-off. sec. $\times k = n$	Power Draught @ 400 cu. ft. per sec. sec. $\times 400 = 0$	Total Draught $o+m = p$	Net Volume cu. ft. $n-p = q$
Month.	(n)	(o)	(p)	(q)
June......	465,393,600	1,036,800,000	1,249,372,800	—783,979,200
July.......	398,465,600	1,071,360,000	1,311,811,200	—913,345,600
August....	1,548,061,400	1,071,360,000	1,130,601,600	+417,459,800

(2) A profile of the river bed at the site of the dam is prepared showing all the information possible relative to the proposition. Each boring made to determine the character of the bottom must be indicated. A plan view showing the location of the borings, contour lines, etc., should also be shown.

(3) A contour map of the entire drainage area should be made showing not only the overflowed area, but also the location and name of each piece of land affected.

(4) Where the contour map is made a drawing showing the levels from the dam to the end of the reservoir will not be required, otherwise it will.

(5) Several good photographs should be taken showing important objects, especially both banks at the site of the dam. These can be used for the prospectus.

When there are no run-off reports for the river, which is under consideration, the best plan is to select some river having a similar drainage basin and situated in about the same part of the country, for which reports have been prepared, and use these reports as outlined below to determine the run off of the river to be developed.

TABLE XIX.

APPROXIMATE EFFICIENCIES USED IN ESTIMATING THE NET POWER AVAILABLE.

Generators	95%	to	90 % net.
Step-up Transformers	97%		92.1% "
Transmission line	95%		87.5% "
Step down Transformers	97%		84.9% "
Distribution to Sub-stations	93%		79.0% "
Rotary Converters	90%		71.1% "

Some branch of the river to be developed for which rainfall data is available, is selected and the run-off measured for each month. The ratio between run-off and rainfall being established for this branch, the rainfall for the driest year is multiplied by this ratio in order to obtain the run-off of the branch.

At the same time the run-off of this branch is compared with that of the similar river, the average of two gauging stations Y and Z being taken, and another ratio determined · then multiplying the flow of the branch for the dry year by this second ratio the approximate flow of the river is obtained.

Tables XV to XVIII show the method of computing the approximate run-off of a river when that of a similar river is known, the evaporation being taken into account. From these four tables curves may be plotted showing the level of the water in the reservoir each month for one or more years. Two of the driest years may be taken.

Another table may be prepared in connection with the ones given herewith, which will show the area of the reservoir for every foot of elevation between the points of maximum and minimum head.

FORM OF REPORT.

Having compiled the foregoing data the report might take the following form, subject to those modifications which each particular proposition will make necessary:

Introduction.

Dear Sir: I beg to submit, etc.

General Description.

The site of the proposed development, etc.

Flow of the Stream.

Curves relating to the measurements made by engineers and Government. Table developing the " ratio " of run-off.

Table giving run-off for stream under consideration.

Table giving evaporation. Table giving net run-off for use through turbines, etc.

Power Capacity of the Stream.

Table giving the efficiencies of each mechanism commencing with the turbine and ending at the point where power is sold.

Curves showing the delivered power for each year when measurements were taken.

Market for the Power.

Table or curves showing the load of the various customers, the rate of increase of the power used by these companies for a few years back and the estimated power which they will use for a number of years in the future. Curve showing the character of the probable daily load.

Auxiliary Steam Operation.

Curve showing the power in the river during a dry year and a shaded portion showing the part of the load which would have to be carried by steam. Table showing the per cent. of

TABLE XX.

Maintenance and Depreciation.

	Maintenance.	Depreciation.
Buildings	1.0	1.0
Auxiliary Mechanism	1.0	1.0
Exciters	5.0	2.0
Storage Battery	5.0	2.0
Generators	2.0	5.0
Transformers	2.0	1.0
Station Wiring	1.0	5.0
Lightning Protection	15.0	0.8
Transmission Line	10.0	0.0
Turbines	3.5	18.0
Gates and Racks	1.0	7.5

TABLE XX-A.

Cost of Operating Steam Power.

Year to which figures are assumed to apply	1905	1910	1915
Total yearly demand at ( ) per kw-hr.			
Average output rate for year kw			
Average output rate for year (max. day) kw			
Engine and generating capacity kept ready for use (h.p. nom.)			
Boilers retained in station			
Per cent. of demand required by steam			
Total output by steam for year kw-hr.			
Cost of coal for actual running at ( ) per kw-hr.			
Cost of coal for banking (at 10, 8, 6, 4 and 3%)			
Emergency labor (.0075, 0070, .004 cts. per kw-hr.			
Permanent force at station			
Maintenance of boilers at 80c. per h.p.			
Maintenance of engine at 60c. per h.p.			
Total cost of operating steam auxiliary			
Cost per kw-hr. output			

steam operation based on the load for a few years back and covering a number of years in the future.

Back Water Conditions.

Full description with curves showing the duration, etc.

Pondage.

Table giving cubical contents of reservoir for each foot of depth the pond will be drawn down.

Curves of the power with shaded portion showing the part of the load the stored-up water will carry (similar to the steam load).

Detailed Description of the Work.

Power house, transmission lines, canals, head gates, dam, miscellaneous.

Estimates of Cost of Construction.

Dam, power house, etc.

Operating Charges.

Maintenance and Depreciation.

Table showing per cent. of depreciation and maintenance on each detail of the work; labor and small supplies. (Table XX.)

Cost of operation of auxiliary power, taxes, interest, office expenses, etc. (Table XXA.)

Revenue.

Full reasons for believing power can be sold at the given price. etc.

Summary.

Gross receipts........................$..................

Operating expenses..................$..................

Net profit..........................$..................

Interest on the investment...........$..................

As an appendix the report may contain small scale drawings of the dam, power house, etc. Photographs made from the tracings are good for this purpose.

CHAPTER IV.

MATERIALS.

Before taking up hydraulic construction it is well to consider the relative suitability of the various materials. Many times structures are subjected to alternate exposure of air and water, and this condition is a severe one which comparatively few materials can successfully withstand.

Wood.

It is a well known fact that practically all woods, if submerged in water, will be preserved from decay. Therefore, where possible, all timber should be submerged. The condition most conducive to decay is that of continual change from dry to wet and wet to dry. The most rapid decay ever witnessed by the writer was where the posts of a flume came in contact with gumbo, a soil common along western rivers, and at the water line. In this case 8x8-inch pine timber lasted only six years. Ordinarily, pine timber under most unfavorable conditions, lasts from eight to twelve years.

With few exceptions, all timber decays first in the sapwood. Hence specifications should exclude the sap. Certain woods are wholly unsuited for work in contact with varying degrees of moisture. Some of these are as follows: Elm (except rock elm), soft maple, willow, poplar, baswood, all oaks (except white, pin oak and live oak), spruce, pine and hemlock.

Among the best, and in order of superiority for such work, are the following: Texas and Oregon fir, red and white cedar, "Hart" yellow pine, live oak, white oak, pin oak, white pine (free from sap), beech, spruce pine, tamerack and hemlock.

Hemlock is given in both lists, as it is on the border-line. The upland hemlock lasts fairly well. Yellow pine, when free from sap, makes a very satisfactory material, and the cost is moderate. For most rivers, white oak makes good timber, yet

there are cases on record where the acid in the oak was attacked by some chemical in the water and the timber destroyed in a few years.

No matter what wood is used, or how well it may be dry-seasoned, after it has been exposed to water for a few years, it will shrink. Of course, the more thoroughly it is dried, the less will be the contraction. The first effect of the water is to swell the wood. Plank, when continually wet on one side so that the wood is saturated, will last indefinitely.

Soft wood is worn away less rapidly by running water than is hard wood.

Metals.

Among the cheaper metals, cast iron resists the corroding effect of water the best. Steel corrodes much more rapidly. All metals corrode more rapidly when exposed to running water, and the higher the velocity the more rapid the corrosion. Steel penstocks wear away rapidly and become rough, increasing the coefficient of friction. Nothing thinner than $\frac{1}{4}$-inch should be used.

For work under high heads such as several hundred feet or more the water frequently bores holes through cast iron turbine runners, and nothing but bronze should be used for such parts.

Cement and Concrete.

Five years ago we were in the steel age, but to-day it is the concrete-steel age. Bridges on all the great railways were then built of steel; to-day the best practice points to the concrete-steel bridge.

The price of steel remains at about the same figure. The price of timber is steadily going up, having increased fully 50 per cent. in 10 years. Interest on money is steadily going down. Cement is each year getting cheaper. The tendency of the times is toward more permanent construction. These facts have contributed to usher in the concrete-steel age. The characteristics of steel have been thoroughly worked out. It has high tensile strength, is quite flexible, has good elasticity, and is uniform in all its features.

But cement is not so well understood. In fact, there are as many ideas concerning its proper combinations as there are engineers.

There are two distinct kinds of cement, natural or Rosendale, and Portland. Some of the largest works have used Rosendale, as, for instance, the Croton dam, but in the last few years Portland cement has been made in the United States in much larger quantities and of such splendid quality that the use of any other is no longer advisable.

Frost affects Rosendale cement, and under no circumstances should it be used when frost can reach it. For the interior portions of large monolithic dams and where it is desired to save a few dollars (a questionable policy) Rosendale might be used. The price of Portland has now gained a point where, on account of its superior strength, there is no real economy in the use of cheap cement.

There are innumerable brands of cement made in the United States to-day, most of which are equal to the English and German cements. Among the best might be mentioned Giant Portland, Lehigh, and Atlas.

TESTING.

All cement, no matter of what brand, should be tested before being used on important works.

The following tests are those recommended by the American Society of Civil Engineers in 1902*, and which are without doubt the most authorative we have to-day.

Sampling.

Selection of Sample.—The selection of the sample for testing is a detail that must be left to the discretion of the engineer; the number and the quantity to be taken from each package will depend largely on the importance of the work, the number of tests to be made and the facilities for making them. The sample shall be a fair average of the contents of the package; it is recommended that, where conditions permit, one barrel in every 10 should be sampled. All samples should be passed through a sieve having 20 meshes per linear inch, in order to break up lumps and remove foreign material; this is also a very effective method for mixing them together in order to obtain an average. For determining the characteristics of a shipment

*Report of the American Society of Civil Engineers' Committee on Uniform tests of Cement.

of cement, the individual samples may be mixed and the average tested; where time will permit, however, it is recommended that they be tested separately.

Method of Sampling.—Cement in barrels should be sampled through a hole made in the center of one of the staves, midway between the heads, or in the head, by means of an auger or sampling iron similar to that used by sugar inspectors. If in bags it should be taken from surface to center.

Chemical Analysis.

Significance.—Chemical analysis may render valuable service in the detection of adulteration of cement with considerable amounts of inert material, such as slag or ground limestone. It is of use, also in determining whether certain constituents, believed to be harmful when in excess of a certain percentage, as magnesia and sulphuric anhydride, are present in inadmissible proportions. While not recommending a definite limit for these impurities, the committee would suggest that the most recent and reliable evidence appears to indicate that for Portland cement magnesia to the amount of 5 per cent. and sulphuric anhydride to the amount of 1.75 per cent., may safely be considered harmless.

The determination of the principal constituents of cement—silica, alumina, iron oxide and lime—is not conclusive as an indication of quality. Faulty character of cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions of the constituents. Cement made from very finely-ground material, and thoroughly burned, may contain much more lime than the amount usually present and still be perfectly sound. On the other hand, cements low in lime may, on account of careless preparation of the raw materials, be of dangerous character. Further, the ash of the fuel used in burning may so greatly modify the composition of the product as largely to destroy the significance of the results of analysis.

Method.—As a method to be followed for the analysis of cement that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, of the New York Section of the Society for Chemical Industry, and published in the Journal of the Society, for January 15, 1902, is recommended.

Specific Gravity.

Significance.—The specific gravity of cement is lowered by underburning, adulteration and hydration, but the adulteration must be in considerable quantity to effect the results appreciably. Inasmuch as the difference in specific gravity are usually very small, great care must be exercised in making the determination. When properly made, this test affords a quick check for underburning or adulteration.

Apparatus and Method.—The determination of specific gravity is most conveniently made with Le Chatelier's apparatus. This consists of a flask (D), Fig. 51, of 120 cubic centimeters (7.32

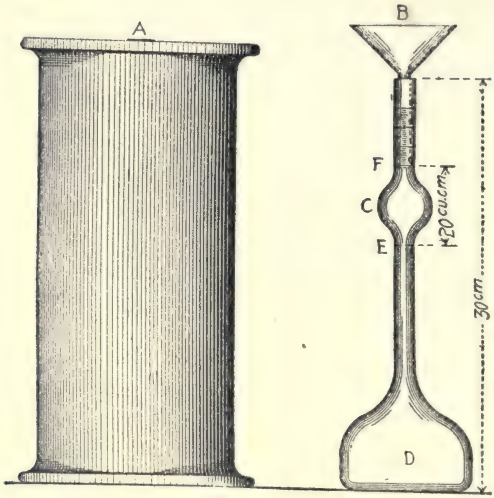


Fig. 51.

cubic inches) capacity, the neck of which is about 20 centimeters (7.87 inches) long; in the middle of this neck is a bulb (C), above and below which are two marks (E) and (F); the volume between these marks is 20 cubic centimeters (1.22 cubic inches). The neck has a diameter of about 9 millimeters (0.35 inch), and is graduated into 1-10 cubic centimeters above the bulb. Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination. The specific gravity can be determined in two ways:

1. The flask is filled with liquid to the lower mark (E), and 64 grains (2.25 ounces) of powder, previously dried at 100° C.

(212° F.) and cooled to the temperature of this liquid, is gradually introduced through the funnel (B) [the stem of which extends into the flask to the top of the bulb (C)], until the upper mark (F) is reached. The difference in weight between the cement remaining and the original quantity (64 grains) is the weight which has displaced 20 cubic centimeters.

2. The whole quantity of the powder is introduced, and the level of the liquid rises to some division of the graduated neck. This reading plus 20 cubic centimeters is the volume displaced by 64 grains of the powder.

The specific gravity is then obtained from the formula:

$$\text{Specific gravity} = \frac{\text{Weight of cement}}{\text{Displaced volume.}}$$

The flask, during the operation, is kept immersed in water in a jar (A), in order to avoid variations in the temperature of the liquid. The results should agree within 0.01.

A convenient method for cleaning the apparatus is as follows: The flask is inverted over a large vessel, preferably a glass jar and shaken vertically until the liquid starts to flow freely; it is then held still in a vertical position until empty; the remaining traces of cement can be removed in a similar manner by pouring into the flask a small quantity of clean liquid and repeating the operation. More accurate determinations may be made with the picnometer.

Fineness.

Significance.—It is generally accepted that the coarser particles in cement are practically inert, and it is only the extremely fine powder that possesses adhesive or cementing qualities. The more finely cement is pulverized, all other conditions being the same, the more sand it will carry and produce a mortar of a given strength. The degree of final pulverization which the cement receives at the place of manufacture is ascertained by measuring the residue retained on certain sieves. Those known as the No. 100 and No. 200 sieves are recommended for this purpose.

Apparatus.—The sieve should be circular, about 20 centimeters (7.87 inches) in diameter, 6 centimeters (2.36 inches) high, and provided with a pan 5 centimeters (1.97 inches) deep, and a cover.

The wire cloth should be woven (not twilled) from brass wire having the following diameters: No. 100, 0.0045 inch; No. 200, 0.0024 inch. The wire cloth should be mounted on the frames without distortion; the meshes should be regular in spacing and be within the following limits: No. 100, 96 to 100 meshes to the linear inch; No. 200, 188 to 200 meshes to the linear inch. Fifty grams (1.76 ounces) or 100 grains (3.52 ounces) should be used for the test, and dried at a temperature of 100° C. (212° F.) prior to sieving.

Method.—The committee, after careful investigation, has reached the conclusion that mechanical sieving is not as practical or efficient as hand work, and, therefore, recommends the following method: The thoroughly dried and coarsely screened sample is weighed and placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward, at the same time striking the side gently with the palm of the other hand, at the rate of about 200 strokes per minute. The operation is continued until not more than 0.1 per cent. passes through after one minute of continuous sieving. The residue is weighed, then placed on the No. 100 sieve and the operation repeated. The work may be expedited by placing in the sieve a small quantity of large shot. The result should be reported to the nearest tenth of 1 per cent.

Normal Consistency.

Significance.—The use of a proper percentage of water in making the pastes* from which pats, tests of setting and briquettes are made, is exceedingly important, and affects vitally the results obtained. The determination consists in measuring the amount of water required to reduce the cement to a given state of plasticity, or to what is usually designated the normal consistency. Various methods have been proposed for making this determination, none of which have been found entirely satisfactory. The committee recommends the following:

Method: Vicat Needle Apparatus.—This consists of a frame K, Fig. 52, bearing a movable rod L, which has a cap A at one end, and at the other end a cylinder B, 1 centimeter (0.39 inch

*The term " paste " is used in this report to designate a mixture of cement and water, and the word " mortar " a mixture of cement, sand and water.

in diameter, the cap, rod and cylinder weighing 300 grains (10.57 ounces). The rod, which can be held in any desired position by a screw *F*, carries an indicator, which moves over a scale (graduated to centimeters) attached to the frame *K*. The paste is held by a conical, hard-rubber ring *I*, 7 centimeters (2.76 inches) in diameter at the base, 4 centimeters (1.57 inches) high, resting on a glass plate *J*, 10 centimeters (3.94 inches) square.

In making the determination, 500 grains (17.64 ounces) of cement are kneeded into a paste, as described in a succeeding paragraph, and is then formed into a ball with the hands, com-

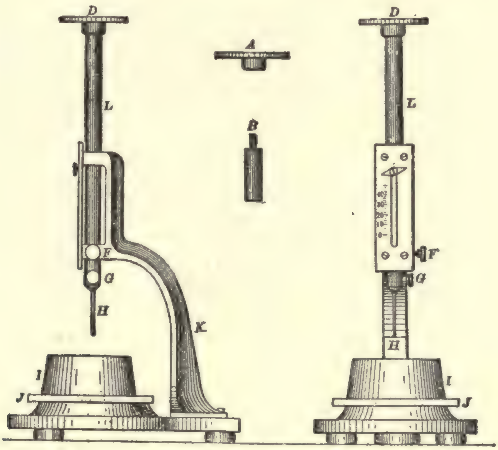


FIG. 52.

pleting the operation by tossing it six times from one hand to the other, maintained 6 inches apart; the pall is then pressed into the rubber ring, through the larger opening, smoothed off and placed on a glass plate (on its large end), and the smaller end smoothed off with a trowel; the paste, confined in the ring resting on the plate, is placed under the rod bearing the cylinder which is brought in contact with the surface and quickly released. The paste is of normal consistency when the cylinder penetrates to a point in the mass 10 millimeters (0.39 inch) below the top of the ring. Great care must be taken to fill the ring exactly to the top.

The trial pastes are made with varying percentages of water

until the correct consistency is obtained. The committee believes that the normal consistency should produce a rather wet paste, since this consistency tends to greater uniformity in the mixing, and since there is less liability of compressing the briquettes during the molding. Having determined in this manner the proper percentage of water required to produce a neat paste of normal consistency, the proper percentage required for the sand mortars is obtained from an empirical formula. The committee hopes to devise such a formula. The subject proves to be a very difficult one, and, although the committee has given it much study, it is not yet prepared to make a definite recommendation.

Time of Setting.

Significance.—The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be fluid and plastic (called the " initial set "), and also the time required for it to acquire a certain degree of hardness (called the " final " or " hard set.")

The former of these is the more important, since, with the commencement of setting, the process of crystallization or hardening is said to begin. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of mixing and molding or incorporating the mortar into the work before the cement begins to set. It is usual to measure arbitrarily the beginning and end of the setting by the penetration of weighted wires of given diameters.

Method.—For this purpose the Vicat needle, which has already been described, should be used. In making the test, a paste of normal consistency is molded and placed under the rod L, Fig. 52; the cylinder and the cap A are replaced by the needle H, one millimeter (0.039 inches) in diameter, and the cap D, the rod L with cap D and needle H, weighing 300 gr. (10.57 ounces). The needle is then carefully brought in contact with the surface of the paste and quickly released. The setting is said to have commenced when the needle ceases to pass a point five millimeters (0.20 inches) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

The test pieces should be stored in moist air during the test; this is accomplished by placing them on a rack over water

contained in a pan and covered with a damp cloth, the cloth to be kept away from them by means of a wire screen; or they may be stored in a moist box or closet. Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point reduces the area and tends to increase the penetration. The determination of the time of setting is only approximate, being materially affected by the temperature of the mixing water, the temperature and humidity of the air during the test, the percentage of water used, and the amount of molding the paste receives.

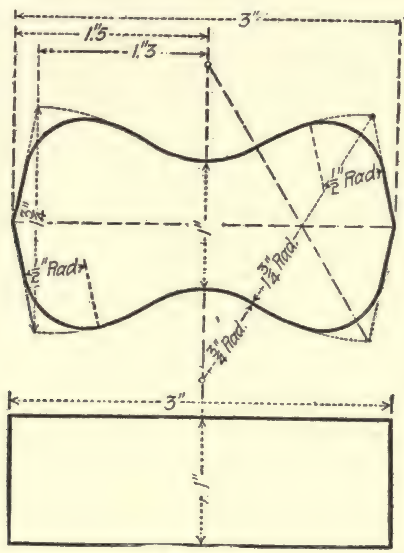


FIG. 53.

Standard Sand.

The committee recognizes the grave objections to the standard quartz now generally used, especially on account of its high percentage of voids, the difficulty of compacting in the molds, and its lack of uniformity; it has spent much time in investigating the various natural sands which appeared to be available and suitable for use. For the present, the committee recommends the natural sand from Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch and retained on a sieve having 30 meshes per linear inch; the wires to have diameters

of 0.0165 and 0.0112 inches, respectively, *i.e.*, half the width of the opening in each case. The Sandusky Portland Cement Co., of Sandusky, O., has agreed to undertake the preparation of this sand, and to furnish it at a price only sufficient to cover the actual cost of preparation.

While the form of the briquette recommended by a former committee of the society is not wholly satisfactory, this committee is not prepared to suggest any change, other than rounding off the corners by curves of ½-inch radius, Fig. 53.

Molds.

The molds should be made of brass, bronze or some equally non-corrodible material, having sufficient metal in the sides to

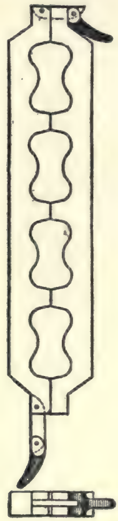


FIG. 54.

prevent spreading during molding. Gang molds, which permit molding a number of briquettes at one time, are preferred by many to single molds; since the greater quantity of mortar that can be mixed tends to produce greater uniformity in the results. The type shown in Fig. 54 is recommended. The molds should be wiped with an oily cloth before using.

Mixing.

All proportions should be stated by weight; the quantity of water to be used should be stated as a percentage of the dry

material. The metric system is recommended because of the convenient relation of the gram and the cubic centimeter. The temperature of the room and the mixing water should be as near 21° C. (70° F.) as it is practicable to maintain it. The sand and cement should be thoroughly mixed dry. The mixing should be done on some non-absorbing surface, preferably plate glass. If the mixing must be done on an absorbing surface it should be thoroughly dampened prior to use. The quantity of material to be mixed at one time depends on the number of test pieces to be made; about 1000 gr. (35.28 ounces) makes a convenient quantity to mix, especially by hand methods.

The committee, after investigation of the various mechanical mixing machines, has decided not to recommend any machine that has thus far been devised, for the following reasons: (1) The tendency of most cement is to " ball up " in the machine, thereby preventing the working of it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine, and (3) the difficulty of keeping the machine clean.

Method.—The material is weighed and placed on the mixing table, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the crater by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for an additional 1½ minutes, the process being similar to that used in kneading dough. A sand-glass affords a convenient guide for the time of kneading. During the operation of mixing the hands should be protected by gloves, preferably of rubber.

Molding.

Having worked the paste or mortar to the proper consistency, it is at once placed in the molds by hand. The committee has been unable to secure satisfactory results with the present molding machines; the operation of machine molding is very slow, and the present types permit of molding but one briquette at a time, and are not practicable with the pastes or mortars herein recommended.

Method.—The molds should be filled at once, the material

pressed in firmly with the fingers and smoothed off with a trowel without ramming; the material should be heaped up on the upper surface of the mold, and, in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the excess material. The mold should be turned over and the operation repeated. A check upon the uniformity of the mixing and molding is afforded by weighing the briquettes just prior to immersion, or upon removal from the moist closet. Briquettes which vary in weight more than three per cent. from the average should not be tested.

Storage of the Test Pieces.

During the first 24 hours after molding, the test pieces should be kept in moist air to prevent them from drying out. A moist closet or chamber is so easily devised that the use of the damp cloth should be abandoned if possible. Covering the test pieces with a damp cloth is objectionable, as commonly used, because the cloth may dry out unequally, and, in consequence, all the test pieces are not maintained under the same condition. Where a moist closet is not available a cloth may be used and kept uniformly wet by immersing the ends in water. It should be kept from direct contact with the test pieces by means of a wire screen or some similar arrangement.

A moist closet consists of a soapstone or slate box, or a metal-lined wooden box—the metal lining being covered with felt and this felt kept wet. The bottom of the box is so constructed as to hold water, the sides are provided with cleats for holding glass shelves on which to place the briquettes. Care should be taken to keep the air in the closet uniformly moist.

After 24 hours in moist air, the test pieces for longer periods of time should be immersed in water maintained as near 21° C. (70° F.) as practicable; they may be stored in tanks or pans, which should be of non-corrodible material.

Tensile Strength.

The tests may be made on any standard machine. A solid metal clip, as shown in Fig. 55, is recommended; this clip is to be used without cushioning at the points of contact with the test specimen. The bearing at each point of contact should be ¼-inch wide, and the distance between the center of contact on the same clip should be 1¼ inches.

Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the briquettes in the testing machine, as cross-strains, produced by improper centering, tend to lower the breaking strength; the load should not be applied too suddenly, as it may produce vibration, the shock from which often breaks the briquette before the ultimate strength is reached. Care must be taken that the clips and the sides of the briquette be clean and free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 pounds per minute. The average of the briquettes of each sample tested should be

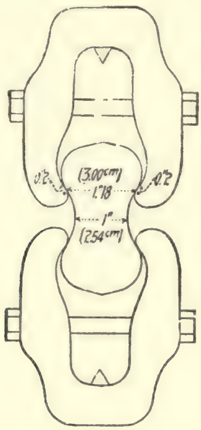


FIG. 55.

taken as the test excluding any results which are manifestly faulty.

Constancy of Volume

Significance.—The object is to develop those qualities which tend to destroy the strength and durability of a cement. As it is highly essential to determine such qualities at once, tests of this character are for the most part made in a very short time, and are known, therefore, as accelerated tests. Failure is revealed by cracking, checking, swelling or disintegration, or all of these phenomena. A cement which remains perfectly sound is said to be of constant volume.

Methods.—Tests for constancy of volume are divided into

two classes: (1) normal tests, or those made in either air or water maintained at about 21° C. (70° F.), and (2) accelerated tests, or those made in air, steam or water, at a temperature of 45° C. (115° F.) and upward. The test pieces should be allowed to remain 24 hours in moist air before immersion in water or steam. For these tests, a pat about 7½ centimeters (2.95 inches) in diameter, 1¼ centimeters (0.49 inches) thick at the center, and tapering to a thin edge, should be made, upon a clean glass plate (about 10 centimeters (3.94 inches) square), from cement paste of normal consistency.

Normal Test.—A pat is immersed in water maintained as near 21° C. (70° F.) as possible for 28 days, and observed at intervals; the pat should remain firm and hard and show no signs of cracking, distortion or disintegration.

Accelerated Test.—(a) A pat is placed on a shelf in a suitable vessel filled with fresh water, but without allowing it to touch the bottom. The water is then gradually raised to a temperature of 45° C. (115° F.) and maintained, at this temperature for 24 hours; or (b), a pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel, for three hours.

To pass these tests satisfactorily the pats should remain firm and hard and show no signs of cracking, distortion or disintegration. Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate. In the present state of our knowledge it cannot be said that cement should necessarily be condemned simply for failure to pass the accelerated tests; nor can a cement be considered entirely satisfactory simply because it has passed these tests.

Submitted on behalf of the committee: George S. Webster, chairman; Richard L. Humphrey, secretary; George F. Swain, Alfred Noble, Louis C. Sabin, S. B. Newberry, Clifford Richardson, W. B. W. Howe, F. H. Lewis.

Simple Tests.

The tests recommended by the Society are quite exhaustive and are those used on very large works where a man is detailed to the testing work alone. For less extensive work, however, a more rapid and less expensive test is desired.

The following simple tests can be made by the engineer himself, with an outfit costing not over $4, and which can be stored in a desk pigeon-hole. The tests thus made will be interesting in themselves, and will be effective and convincing aids in rejecting most bad cements which may be offered, and will also have the preventive effect of causing manufacturers to send their lower grades of cement elsewhere and to send only their best products to the places where such tests are probable:

(1) *For Fineness.*—Sift three to four ounces of cement through a standard test sieve of 100 meshes per linear inch. Reject cement of which 10 per cent. by weight is retained on the sieve. This is conservative, and the limit may be made smaller, for many Portland cements are now in the market which will leave less than four per cent. A test by 200-mesh sieve, with a 30 per cent. limit, is desirable, but takes time.

(2) *For Quickness of Setting.*—Make a pat of four ounces of neat cement, adding one-quarter to one-fifth its weight of water and making a putty-like ball which can be dropped on the table and retain its form without falling to pieces. Press this upon a 3x4-inch glass plate, leaving it ½-inch thick in the center and sloping to thin edges all around. Note time required to take initial set. Reject cement which sets in less than 25 minutes. It may take three hours or more, but it will be better for paving if it sets in one hour. The instant of " initial set " is determined by nothing when the surface will support a 4-ounce weight resting upon the smooth flat end of a 11/12-inch diameter wire.

(3) *For Soundness.*—Use the pat on glass above described and note when it sets enough more to make it difficult to indent it with the thumb nail, or when it will support one pound on the smooth flat end of a 1/24-inch wire, which may be considered as indicating " a hard set." Then put the pat with its glass plate over boiling water until the steam has heated them, and then immerse and keep them in the boiling water for three hours. Reject Portland cement if the pat shows radiating cracks in the center, or shows blow-holes on the surface, or curls up from the glass, or cracks at the thin edges. Good natural cements may fail to endure this test (which is a severe one), and it may properly cause the rejection of some Portland cements which would endure it after being " air-slacked " or " seasoned."

(4) *For Purity.*—Provide a glass-stoppered bottle of muriatic acid, two shallow white bowls or two ½-inch by 6-inch test tubes, a glass rod and a pair of rubber gloves. Put in a bowl or a tube as much cement as can be taken on a nickel 5-cent piece; moisten it with half a teaspoonful of water; cover with clear muriatic acid poured slowly upon the cement while stirring it with the glass rod.

Pure Portland cement will effervesce slightly, and will give off some pungent gas and will gradually form a bright yellow jelly without any sediment.

Powdered limestone or powdered cement-rock mixed with the pure cement will cause a violent effervescence, the acid boiling and giving off strong fumes until all the carbonate of lime has been consumed, when the bright yellow jelly will form.

Powdered sand or quartz or silica mixed with cement will produce no other effect than to remain undissolved as a sediment at the bottom of the yellow jelly.

Reject cement which has either of these adulterants.

Powdered slag mixed with cement unfits it for pavement work. The adulteration is indicated in the dry cement (when coloring matter does not conceal it) by a lilac tint, and it is also indicated on the surface of a test-pat after drying by brown and green and yellow discolorations.

A chemical test will show the presence of slag if made as follows: Provide an ounce of mixture of methylene iodide (CH_2I_2) and benzine, in which the methylene (the specific gravity of which is $3.^{292}$, being the heaviest organic liquid) is reduced to the specific gravity of $2.^{95}$ by addition of benzine. The methylene is uncommon and costs $1 an ounce.

Into a ½-inch test tube put ½-inch of the dry suspected cement and pour in a little of the mixture, stirring to a thin grout Then cork the tube and let it stand. If slag is present, it will remain at the top, while the cement will settle to the bottom. The separation cannot be seen if coloring matter is present.

Coloring matter in any cement will show itself in the acid test by giving a black or gray color to the resultant jelly, which would otherwise be yellow. The coloring matter may or may not be injurious in itself, but its presence shows that the manufacturer wished to disguise the cement, which should be rejected, because there are a plenty of good cements which need no disguise.

Weight.—The several kinds of cement differ materially in weight, and any cement that varies much from these average weights should be examined specially.

The standard barrel contains 3.65 cubic feet, and the standard bag is one-fourth of a barrel. The average weight of a cubic foot of packed cement is: Portland, 104 to 114 pounds; puzzolan, 90 pounds; natural, 75 to 82 pounds for Eastern and 70 to 72 for Western, the average net weight of each per barrel being 375 pounds, 330 pounds, 300 pounds and 265 pounds.

Results.—These tests will be conclusive as far as they go, and will cause the rejection of no good cements. The makers of high-grade cements would not object to these requirements and would not increase the price because of them.

Beam Test.

The best test of all is to construct small beams of the actual materials to be used and then select that cement and mixture which gives the best results. Of course tests should be made to determine the freedom from slag, etc.

For testing use a beam 2x2x24 inches. A heavy timber lever can easily be made for the center load test. Then the formula

$$\frac{P\,L}{4} = \frac{s\,b\,d^2}{6}$$

gives the safe load P. (see Table XXIX)

USES.

During the process of manufacturing the cement is frequently over or under burned, making an inferior quality. This is usually mixed in with the other cement and sold. There are times when this inferior cement is sold to the small buyer with the idea that it will not be tested. It is to discourage such acts that the cement is tested.

Cement which has been over burned sets with great rapidity and there are times when the hydraulic engineer wants just such cement. By sending to the factory such cement can usually be procured, and if wanted in sufficient quantity it will be made especially to suit the requirements. It must be borne in mind, however, that such cement is only about half as strong as the perfect article.

By reading over the tests for cement, the virtues desired in a good cement will be understood.

Cement should be put up in barrels though it adds somewhat to the cost. Paper sacks preserve the cement from loss and moisture better than cloth sacks but are more liable to injury from rough handling. The most common form for shipping is the cloth sack, the sacks being saved and sent back to the factory.

Cement alone, neat cement, is seldom used unless it is for pointing or plastering, the usual way being to mix it with sand and crushed stone; sand and slag; sand and burnt clay or gumbo, gravel, cinders, etc. This added material is called the aggregate. The aggregate must always be clean and, when dirty, must be well washed.

TABLE XXI.

Name.	Weight per bbl. in lbs.	Volume per bbl. in cu. ft.	Volume per bbl. in cu. ft net.
Portland.......	380	4	3.6
Natural........	300	4	3.6

TABLE XXII.

WEIGHT OF CONCRETE.

Cinder concrete................	about	105	lbs.	per cubic foot.
Crushed stone concrete...........	"	140	"	"
Gravel concrete..................	"	150	"	"
Slag concrete....................	"	135	"	"
Cement mortar 1-2..............	"	116	"	"

The proportions of the aggregate and cement depend not only on the aggregate but also on the character of the work for which it is used. Here is where the judgment of the engineer is brought into play.

The walls for flumes, penstocks, canal lining, floors, etc., should be in the proportion of 1½ barrels Portland cement to the cubic yard of gravel having the proper proportion of sand and gravel, or if crushed stone is used, the proportion, one cement, two sand and four stone is good practice. For less important work one barrel of cement to the cubic yard of gravel and 1-3-6 if stone is used.

The amount of water used in mixing is one of the open questions. The best engineering practice, however, outside of the laboratories, seems to be to use enough water so that when the concrete is tamped into the forms water stands on the surface and the whole mass quakes when tamped.

A wet concrete is more apt to be well made than a more dry

TABLE XXIII.

Concrete Aggregates.

Cement.	Sand.	Gravel.	Crushed Stone.	Character of Work.
1		6		Culvert sides and bottom.
1		5		Culvert arch.
1		4		Culvert arch especially strong.
1	1			Water-tight under high pressure.
1	2 or 2½		4	Water-tight under high pressure equally good.
1	2½		5	Penstocks, lining for reservoirs.
1	3		6	Generator and building foundations.
1	2		3	Lining for reservoirs.
1	3		6	Backing for reservoirs.
1	3		5	Piers and abutments.
1		7		Steel concrete bridges 25 foot to 35 foot span.
1	2		4	Steel concrete bridges arches span 40 feet to 60 feet.
1		6		Sewers.
1		7		Large breakwater.
1		3		Steel concrete piling, 5000-pound hammer.
1	2½		5	Floor slabs.
1	2		4	Beams.

mixture. Special work which is under the eye of the engineer can be done with the minimum amount. Numerous experiments have proven that a wet concrete gets practically as strong as the more dry mixture.

In Table XXIII are given some of the mixtures used on important works, actually built, in the United States.

SAND CEMENT.

In the West, when, owing to high freight rates and difficulties of transportation, the price of cement reaches a high figure, the conditions are such that sand cement demands recognition.

F. L. Smidth is the inventor and a royalty of 10 cents per barrel is charged.

The silica sand is placed in a revolving drum in which are pebbles of great hardness. These pebbles grind the sand and equal volume of cement up into a much finer dust than was even the cement before it was put in. The combined mixture of half sand and half cement is then assumed as being all cement, and used with the usual proportions of gravel or sand and stone.

Experience with it in California indicates that it gives good results.

The following is the itemized cost of a barrel of sand cement, given in *Water Supply and Irrigation:*

One-half barrel Portland cement	$5.00
One-half barrel sand	.18
Grinding sand	.20
Royalty	.05
Total cost of 375 pounds sand cement	$5.43

The above was ground so that 95 per cent. passed a 180-mesh sieve.

Using 340 pounds of the above per cubic yard of concrete we have the following cost per cubic yard of concrete:

Cement sand	$4.93
Sand	.50
Crushed rock and gravel	2.50
Labor	1.00
Total	$8.93

BURNT CLAY AND GUMBO.

The engineer is often called upon to do concreting where there is no stone or gravel. In such localities there is usually clay or gumbo which may be burned and used in the place of the broken stone. The clay or gumbo is burnt as follows: Cord wood is piled in a pile, say 12x12x1 foot. On this spread a layer of coal or slack about four inches thick, and on top of all 15 to 20 inches of clay or gumbo.

On firing the wood enough air enters the pile to enable slow combustion without vitrifying the material. This process costs from 25 to 40 cents per cubic yard. Shrinkage of these clays is about 12 per cent. during burning, and the crushing strength of the burnt product is often as high as 400 pounds per square inch.

Gumbo is a black, sticky mud, found along most of the rivers of the United States, especially in the Central and Western States. It is now being used to quite an extent for railroad ballast and highways.

Table XXIV shows how the various items of expense are distributed. Of course, the items of labor, forms, mixing and placing, will vary with every case. The costs are given in dollars per cubic yard.

TABLE XXIV.
COST IN DOLLARS OF CONCRETE WORK PER CUBIC YARD.

Character of the Work.	Labor and General Expenses.	Forms.	Lumber in Forms.	Mixing and Placing.
Power house walls. Surface finished in rock work	.20 to .30	.60 to .75	.40 to .60	1. to 1.5
Power house walls. Surface rough	.20 to .25	.50 to .60	.35 to .50	1. to 1.25
Foundations for buildings, generators, etc.	.15 to .20	.15 to .25	.10 to .12	.8 to 1.00
Canal slopes and bottoms (filling)	.10 to .20	.10 to .15	.01 to .05	.8 to 1 25
Canal slopes and bottoms (surface)	.12 to .25	.25 to .35	.01 to .10	1.25 to 1.5
Walls having numerous windows, etc. Fancy work	.25 to .40	1.50 to 2.0	.75 to 1.00	1.5 to 1 75

COSTS.

A few years ago the idea obtained that concrete should cost at least $6 per yard, but experience has robbed concrete of all its mystery and we must accept it as the best friend the engineer has to-day.

In 1903 the author built a large power-house, the concrete for which cost as follows:

Hand Mixed	All labor (hand-mixing)...	$0.75 per cubic yard
	Gravel..................	.25 " " "
	Labor on forms..........	.68 " " "
	Cement, 1½ barrels........	2.52 " " "
	Total................	$4.20 " " "

The forms were built of the plank and timber used in the construction of the dam and so cost nothing. The outside of the power-house was finished to represent coursed masonry. The concrete was all hand-mixed, the gravel being dumped from wagons holding just one cubic yard, directly upon the mixing platform. This cost is for the concrete tamped in place, and allows for all shrinkage from the batch measurements.

HAND-MIXED CONCRETE.

In mixing by hand or machine the crushed stone should be well washed off by means of a stream of water as it comes on to the platform. If wheel-barrows are used they should be of steel and have numerous holes drilled through the bottom to allow the water to drip off. The cement should not be taken from the sack to measure, simply allow 9/10 cubic feet per sack of

TABLE XXV.

Sizes of Gauge Boxes.

Proportions.	Sand Box.		Stone Box.	
	Size.	Vol. cu. ft.	Size.	Vol. cu. ft.
1—2½—4	2′ 9″×2′×1′ 8″	9.25	5′×4′ 5½″	14.80
1—3 —6	2′ 9″×2′×2′ 0″	11.10	5′×6′ 8″	22.20
1—2 —5	2′ 9″×2′×1′ 4″	7.40	5′×6′ 6⅝″	18.50
1—2½—6½	2′ 9″×2′×1′ 8″	9.25	5′×7′ 2½″	24.05

95 to 100 pounds. If gravel is used it should not be washed unless the gravel has more than 10 per cent. by weight of loam or 15 per cent. of clay, as it has been found that up to these proportions the strength is improved by the loam or clay.

When the mixing is in a cramped-up place where wheel-barrows have to be used and the foreman is of the second class, the best plan is to have the wheel-barrows dumped into gauge boxes. Table XXV gives the sizes of some gauge boxes found convenient for the proportions given. The sand box is placed on the platform and filled level full. The desired amount of cement is then mixed with the sand, the box having been removed, and alongside the sand cement the stone box is filled with the washed stone, the box removed and the cement sand mixed with it once over, the necessary water being played on it through a garden sprinkler.

The more the concrete is mixed the better, but the above mixing ensures good work.

For arch construction use fine crushed stone or gravel, of an even size, not to exceed one-inch grade. To determine the exact mixture, take a vessel full of stone or gravel and fill the space in same with sand, by shaking the sand into the stone until the bulk begins to enlarge, showing that no interstices remain unfilled, then measure the proportions of sand and stone. Use one portion of Portland cement to three portions of sand, and proportion of crushed rock as the test may determine.

Beams are always tension, and the floor above acts as the compression member, consequently the highest quality of tension concrete is required, which is gravel or fine crushed stone of not over one-half-inch grade.

Cinders are not as valuable for beam as for floor or arch construction, where their lightness is a consideration.

Under ordinary conditions, unless exposed to excessive heat or excessive rain, the following is a safe table for the strength of concrete:

TABLE XXVI.

When	30	days	old	60	per cent.	of full	strength
"	60	"	"	75	"	"	"
"	90	"	"	85	"	"	"
"	120	"	"	90	"	"	"
"	180	"	"	95	"	"	"
"	360	"	"	100	"	"	"

The extent to which concrete is hurt by freezing can be ascertained by estimating that after freezing it will not develop more than 40 per cent. of the balance of the strength it would have attained had it not frozen.

The most economical and expeditious method for hand mixing is to dump the gravel or sand and stone directly from the wagons holding one cubic yard on to a large mixing board having an area of about 24x32 feet.

If stone and sand are used for the aggregate, two wagon loads of stone are dumped, making two piles in line on the board. A wagon having a fixed partition extending down the middle of the wagon box, and loaded so that one-half of the load of

sand is just right for the cubic yard of stone, is then driven over the stone piles and a half dumped on top of each pile. Four men with square pointed shovels (two at each end of the pile) then commence at the ends and turn the sand and stone over, working toward the middle. When it is all turned over the board men turn around and work the pile back into its former position. The cement is now scattered over the top of the pile by the man who usually acts as a sub foreman. The board men then repeat the first mixing operation, water being sprinkled on the mixture in the meanwhile, through a garden sprinkler. Wheelbarrows take the concrete away, having been filled by the board men. The board men should be the pick of the workmen and should receive 10 per cent. more pay. Three batches

FIG. 56

should be run all the time in order to get the most efficient results, and all the men should be employed necessary to take care of every detail of the work. It is very poor policy to skimp the help in concrete mixing.

An ordinary No. 4 tank pump, worked by two men, will raise enough water 20 or 30 feet to wet 100 cubic yards of concrete per day.

If the concrete is made of sand gravel the first mixing is, of course, unnecessary, and the cement is spread directly over it and mixed twice over by the four board men.

MACHINE MIXED CONCRETE.

On large work it becomes necessary to handle the concrete in such bulk that hand mixing becomes too slow and expensive.

For a small job the cost of transporting and setting up a mixer makes the cost more than if done by hand, but when thousands of cubic yards are made the machine mixed concrete is much cheaper. There are many forms of mixers on the market, though all may be divided into two classes, *gravity* and *mechanical.* The gravity mixer (see Fig. 56) depends on the force of gravity to mix the aggregate as it falls down a spout lined

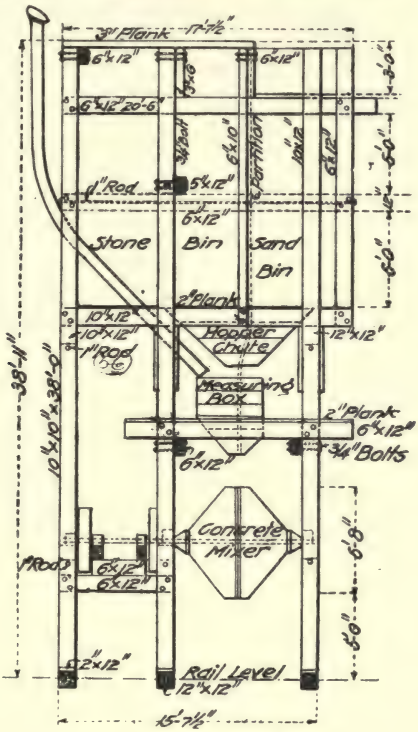


FIG. 57.—Concrete plant.

with projections which deflect the concrete from side to side, and thus mix it. These mixers are quite cheap and simple, but there is reason to believe that they do not give as good results as do the mechanical mixers, though they compare favorably with the usual hand mixed cement. Of the mechanical mixers the cubical mixer is one of the best. It consists simply of a square metal box mounted on an axis passing through two opposite corners and revolved by steam, electric

gasoline or other power. The proper amount of water is fed into the box through a pipe and the batch is dumped in through a door. Where the work to be done is extensive it will pay to fix up a good concrete plant. In Fig. 57 is shown an elevation view of a mixing plant. A large bin is provided for the stone and sand. Under the bin is a measuring box having a movable partition so that any proportion of sand and stone can be obtained. The cement is dumped into a sheet iron chute which empties into the measuring box. The measuring box dumps into the mixer through a canvas tube. The mixer is placed high enough above the ground so that cars or one horse carts may pass under for filling.

All mixers into which a certain amount of aggregate is dumped, mixed and drawn off after the motion of the machine has stopped, are called batch mixers. That is a batch is mixed and emptied and then another is put in.

To avoid the delay of waiting to fill and dump, continuous mixers are sometimes used. In these mixers the proper aggregates are fed in at one end of the mixer continuously, and taken out at the other end.

It is generally conceded that machine mixed concrete is the best. The cost of mixing concrete by machine is from 50 to 70 cents per cubic yard. This includes all machine expense and placing the concrete in the forms, but does not include the forms, tamping or cost of material.

FORMS.

Next in importance to the mixture of the concrete is the building of the forms. It requires the constant vigilance of the engineer to produce good results with the class of labor usually to be procured. The forms must be so built that there will be no springing of the plank. When the work is rushed the green concrete may be several feet deep in the forms and the top being constantly tamped, causes great pressure on the sides. If, after the concrete has partly set, the forms spring ever so little, the concrete assumes a new position and a large part of its strength is gone.

On large engineering jobs there is usually a great amount of rough lumber used in which case the forms may be built of it, using the heavy timber for posts and the two-inch plank surfaced

on one side and edged for the sheeting. It must be remembered that the concrete will reproduce the finest cracks and grain in the wood sheeting, so great care must be taken to have the surface perfectly smooth and level. The forms should be washed with soft soap just before filling and any cracks or rough places should be filled with hard soap, putty, or any other filling which will not discolor the concrete. Opposite posts should be tied together by means of soft iron wire passed through several times, then twisted up tightly. This is the safest and cheapest method of stiffening. One-half inch bolts may be used but they cost more and are more difficult to obliterate from the exposed surface of the finished concrete.

Arches are formed over centers something like that shown

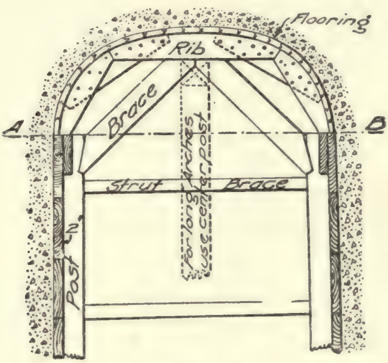


FIG. 58.

in Fig. 58. No part of the form work should be more rigid than the centers. Plenty of bracing should be used especially when the timbers can afterwards be used on the works so that their only cost is the putting in place. The center above the springing line A B, is built in any convenient place and then carried out and stood upon the posts. The ribs should not be more than two feet apart.

Great care must be taken not to remove the forms too soon. A common rule is to allow them to stand nine days, but while this is longer than necessary for some work, it is not enough for other. A long arch should stand two or three weeks, while thin walls only meant to maintain a vertical pressure, can be uncovered in three or four days if necessary.

ROCK WORK.

Plain concrete looks too much like plaster to present a good appearance, so the surface is usually made to resemble rock work by tacking strips to the sheeting as in Fig. 59. The appearance thus procured will add from three to four cents per square foot to the cost of the form but will be worth much more than that where a fine appearance is desired.

Arches may be given the proper appearance as shown in Fig. 60. The strips used to form this rock work are surfaced all

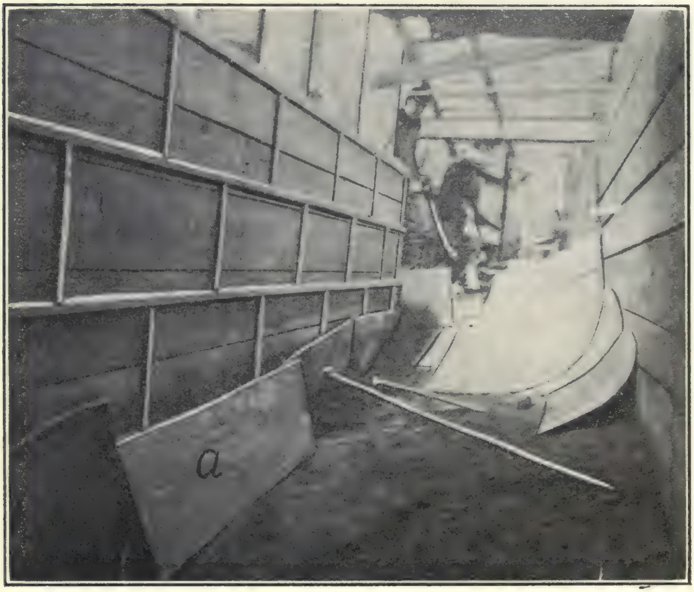


FIG. 59.

over and must be of clear stuff, the size may be made to suit the taste, but the section given in the lower part of Fig. 60 gives good results.

SURFACING.

Where a fine appearance is desired the following method of surfacing is used: As soon as the forms are removed and *while the concrete is somewhat soft*, the surface is rubbed with pieces of grindstone or blocks of concrete having handles moulded in and composed of one part cement to two parts sand. The sur-

face is then wet down; washed with a coat of grout, of one part cement to one part sifted sand, and rubbed with a wooden float. This finish will not peel and is hard and of uniform color. The concrete may be pricked or tooled to make it more nearly resemble real rock.

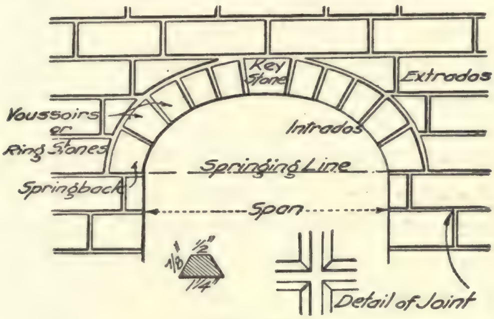


FIG. 60.

Use sheets of steel, called facing boards, for the purpose of holding the coarse concrete away from the surface which it is desired to finish, and have thin grout of one part cement to two parts sand filled in between the facing boards and the form as shown in Fig.61.

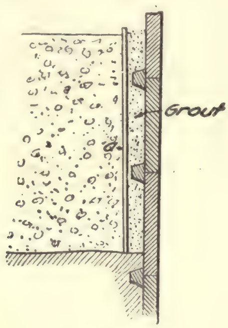


FIG. 61.

When strips are used to get the rock-work effect, the facing board is merely a sheet of iron or steel with holes in the top corners to admit of their being lifted. But when the surface is plain the facing boards have ribs attached as in Fig. 62 to keep them an inch or so away from the form.

The size of the boards will depend on the size of the walls and usually several lengths are used. The width should be about 12 inches.

A coal scuttle having a straight edged snout is handy for pouring the grout in between the boards and the form.

Where possible, build the forms up to full height at the start

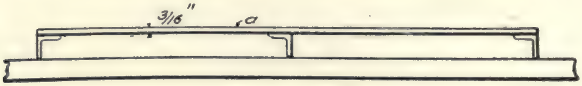


Fig. 62.

and dump the concrete at all stages of the work from the top level. In hydraulic work this can usually be done as the embankments have to be built to the top levels and the mixing boards may be placed on the embankment. The necessary shoveling of the concrete back into place counteracts the tendency of the concrete to unmix in falling

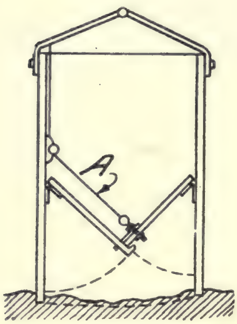


Fig. 63.

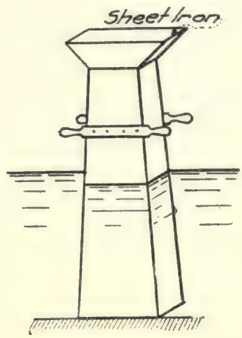


Fig. 64.

CONCRETE LAID UNDER WATER.

Concrete properly laid under water will have a greater strength than if laid in the air. All currents must be avoided, and if this is impossible, the mixture must be enough richer in cement to allow for that washed out. There are several methods for depositing concrete under the water, the best of which is shown in Fig. 63. A large square, steel or wood bucket is made having trap doors as shown. By pulling the wire A the bottom is let

down and the concrete deposited. The box may hold as much as a cubic yard or more of concrete and is handled by means of a derrick. A tube as in Fig. 64 is often used on small work or even a canvas bag, the idea being to get the concrete on to the bottom without washing out the cement. Concrete simply dumped into the water, unless it is very rich and the water shallow, is worthless.

When the current cannot be stopped concrete may be deposited in sacks partly filled and tamped with a heavy tamp. A large piece of canvas held against the current and the concrete deposited against it often keeps one out of a serious difficulty.

GENERAL REMARKS.

In depositing the layers of concrete in the forms it is necessary to keep the courses level, otherwise the facing boards

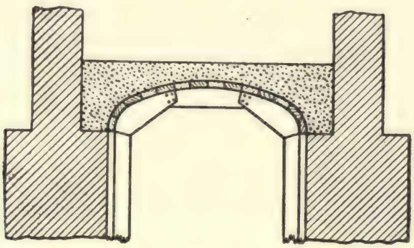


FIG. 65.

cannot be successfully used and the exterior finish will show scars. Courses should be run clear across arches as in Fig. 65. Concrete begins to set the moment it is wet and the quicker it is placed in the form, and the less it is disturbed when so placed, the stronger will it be. Thus, if the forms spring after a few hours of setting the strength is greatly impaired. No blasting should be done near the concrete till it has set for at least two to five days.

Before each course is laid the preceding layer must be *thoroughly* swept off and *wet down*. All surfaces between layers must be left as rough as possible, and where a first class job is desired these partly dried surfaces after being wet down should be coated with a thin layer of grout of one part cement and two parts sand. This greatly increases the strength of the joint.

If concrete is deposited and left exposed to the sun, the result will be that at least one-half an inch of the top surface will be absolutely dead and will form a serious parting line in the wall, as the next layer will not adhere to it. Keep the surfaces wet, covered with a damp canvas, straw, etc.

The edges of the coping may be rounded with a Crafts edger or with a wood fillet and the joints struck with a Crafts jointer.

PECULIARITIES OF CONCRETE.

Concrete expands from heat and about the same amount from absorption of moisture.

The deck of a reinforced concrete dam will expand about $\frac{1}{4}$-inch per 100 feet when the water is raised upon it.

An iron rod embedded in the concrete is gripped firmly by the contraction and the average resistance to pulling out is about 500 pounds per square inch of surface.

Anchor bolts grouted in with neat cement will resist extracting better than if set in lead or sulphur.

The sun shining on a concrete pier tends to warp it.

Slag cements have not yet been proved reliable for hydraulic work.

Natural cements should never be used, especially where exposed to frost.

Concrete has no safe tensile strength.

Concrete allowed to set in a pipe contracts so that it may be shaken out.

Good concrete contracts or expands $\frac{1}{200,000}$ for each degree Fahrenheit change in temperature.

A concrete strut placed between two unyielding abutments will set up a pressure within itself of 15 pounds per square inch for each change of one degree Fahrenheit temperature.

CONCRETE IN FREEZING WEATHER.

It frequently becomes necessary to lay concrete in freezing weather. If the concrete freezes its strength is less than half what it would otherwise be. To prevent freezing the water used is made quite salt. Barrels half full of salt are kept full of hot water and frequently stirred up. All the water used is taken from them. The concrete is then rushed into the forms

and covered with a canvas and a layer of manure or compact straw or hay. If deposited in water, the water keeps out all frost. If freezing cannot be prevented use plenty of cement and salt. Salt does not affect the strength of the concrete.

Concrete so made sets very slowly and is dangerous where it is to sustain pressure in less than two or three months time. Large fires built on the windward side will keep the temperature below freezing. Often a large tent may be placed over the work during construction, and heated by means of large piles of cordwood kept burning constantly.

A tent 150 feet long and 60 feet wide can be bought for from $500 to $600.

Concrete must not be placed on frosty steel reinforcing as the concrete drops away on the under side when forms are removed.

TABLE XXVII.

EFFECT OF AGE AND FROST IN STRENGTH OF CONCRETE.

Age. days.	Strength pounds per square inch.	Remarks.
9	213	Tested in usual way.
28	275	Tested in usual way.
7	123	Tested in dry room.
28	130	Tested in dry room.
7	80	Frozen every night and
28	100	thawed every day.
7	88	Frozen all the time.
28	108	Frozen all the time.

There is no absolute necessity of using salt. Shelters should be placed over mixer and stoves placed inside the structures.

Hot water should be used and the sand heated.

As far as could be determined without actual tests, the salt does not retard setting, as where the concrete was kept heated it set quickly.

After the concrete has been deposited it should be immediately covered with tar paper and over the paper should be spread about 10 inches of manure.

As long as the interior of the structures is kept heated the concrete on the exterior and next to the forms will not freeze.

The concrete under the tar paper will not freeze and will get very hard.

Exposed concrete which can not be heated from within can be encased with manure outside the forms.

It has been found convenient to provide a trap door in the outside lagging so that the setting of the concrete could be watched.

Some authorities claim that Improved Union cement is the best to use in cold weather. One rule is that slow setting Portland must not freeze in less than four days after placing, and quick setting can freeze in 12 hours if kept frozen till set.

Some interesting experiments were recently made on slag cement (not slag Portland) with the results shown in Table XXVII.

Concrete-Steel.

Concrete possesses the qualities of permanence and great crushing strength but little or no tensile or shearing strength. It is the purpose of steel reinforcing to give to the concrete these two items of strength which it lacks. The ideal reinforcing is such as to form a beam which will always fail at the center by pulling apart the steel bars, and at no other part. When concrete fails by shearing it fails between wide limits and a large factor of safety is necessary, but when all the tension is carried by the reinforcing, the factor of safety need be only such as is used for steel work.

Scientific reinforcing does not teach the filling up of the concrete with large steel beams as is sometimes done, but to distribute the proper amount of comparatively small rods throughout the mass and in such a way as to take up all tensional moments. Steel expands .0000064 part of its length for each degree Fahrenheit, while concrete expands about .0000057 of its length for each increase of one degree Fahrenheit. This means that in the case of a steel beam 20 feet long imbedded in concrete, there will be a difference in the expansion and contraction from maximum to minimum temperatures, of about 1/16 inch, Though this is a small amount, it is enough where the reinforcing is large beams, to destroy all adhesion between the steel and concrete. Concrete shrinks on setting about $\frac{1}{1000}$ part of its dimensions. Therefore, if there is a large I-beam

imbedded in its mass as in Fig. 66 there is a tendency to form a crack as shown. The crack does not necessarily occur, but a strain is set up which weakens the wall. For these reasons

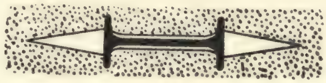


FIG. 66.

large steel beams should not be used for reinforcing, but some of the many patent reinforcing bars made especially for the purpose. The Ransom bar is one of the best known and consists of a square bar, say one inch square twisted many times.

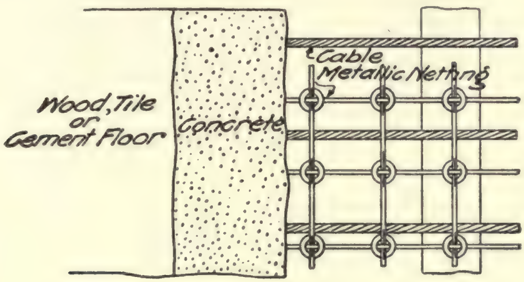


FIG. 67.

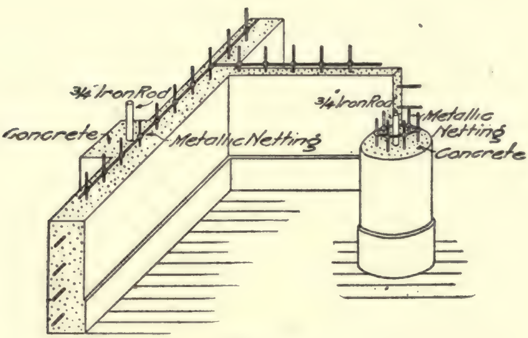


FIG. 68.

The International Fence and Fireproofing Company of Columbus, O., make a stranded cable which is used in connection with woven wires as in Figs. 67-68.

The Trussed Concrete Steel Company of Detroit, Mich., have perfected a system of reinforcing which is undoubtedly one of the best we have. They have conducted numerous tests o. beams and slabs which should be of great value to the engineer. It will be seen that the heavier beams are about half as strong as steel beams of the same depth.

Figs. 69-72 show a few of the uses for the reinforcement. Fig. 73 shows the bar.

The cost of the International cables is about $2\frac{1}{2}$ cents per foot, and their metallic sheeting costs 3 cents per square foot. The

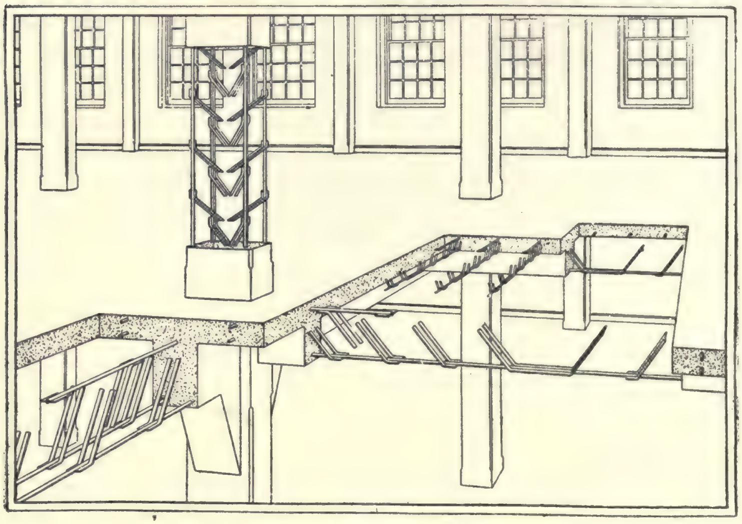


Fig. 69.

Kahn bars cost about 3.9 cents per pound for the small bars, and 3.25 cents for the larger.

In Fig. 74 is shown a type of hollow concrete steel construction (Ransom) which possesses many valuable features. Any form of reinforcing may be used. The cost is about 25 to 35 cents per square foot of exterior surface.

Fig. 75 shows one method of forming the air cells a, a', etc. The corner curve posts B are covered on the sides c, c', with sheet iron and are not fastened to the rest of the form. The two parts D, D' are separate from the posts. The brace E is a piece of 2x6-inch timber, and two are used to hold each form in place.

The forms are about four feet high. The brace is removed first and the posts pulled out when the entire form easily comes out. The posts are allowed to project enough above the form to permit a chain being attached for pulling them out.

At Charles City, Ia., the writer has just completed a reinforced concrete penstock, 1100 feet long and having a capacity of 18,000 cubic feet per minute. The loss of head will be 12 inches.

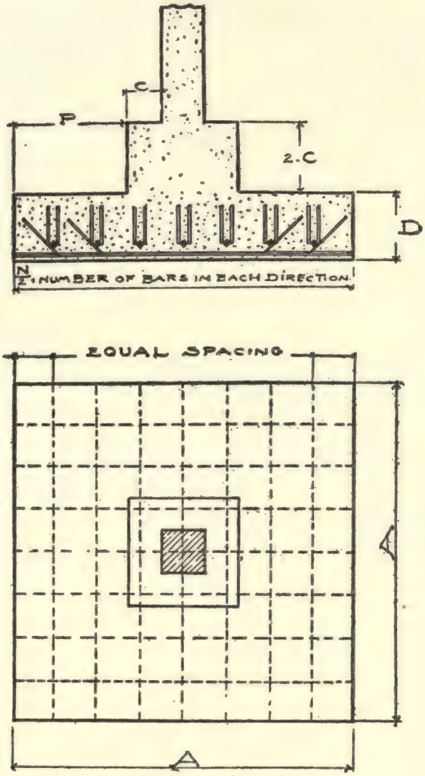


FIG. 70.

Fig. 76 shows how the forms were constructed. The lagging was $\frac{7}{8}$ inch flooring, 3 inch being used on the curves. The sections were 12 feet long and had five ribs per section. Fifteen sections were built which made 182 feet of penstock. The outer forms were made of 2 inch surfaced plank and 6 by 8 inch posts, on 4 foot centers.

As the work progressed many improvements were made in the inner forms and these are shown in the figures.

It was found to be very difficult to clean the bottom at *d*, and the bottom forms, *e*, were made so that they could be moved in toward the center 8 inches and without disturbing the upper forms resting on the posts, *f*.

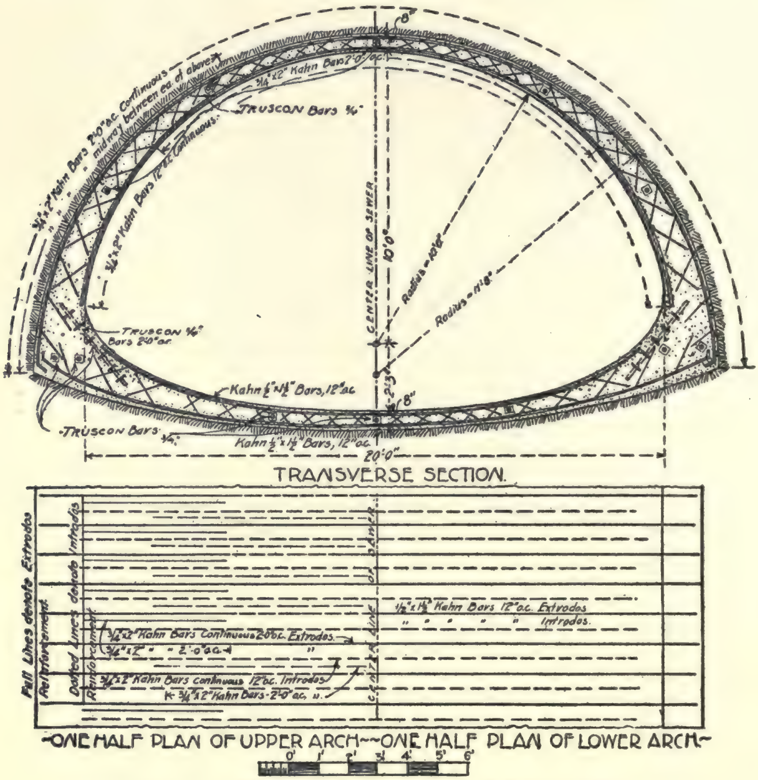


FIG. 71.—Penstock reinforced with Kahn Bars.

To take out the forms the lower forms were moved toward the center and removed. Then one half of the upper forms was taken out by dropping the side at *g*, first.

The section to the left was that built where the ice would pound and the half on the right where the penstock passed into the excavation.

Where the penstock left the cliff it was carried on three

masonry piers, but where it was rock up to the bottom of the penstock it was built as shown. On the side where there was no rock cliff dowel pins were set as shown. The rough surface

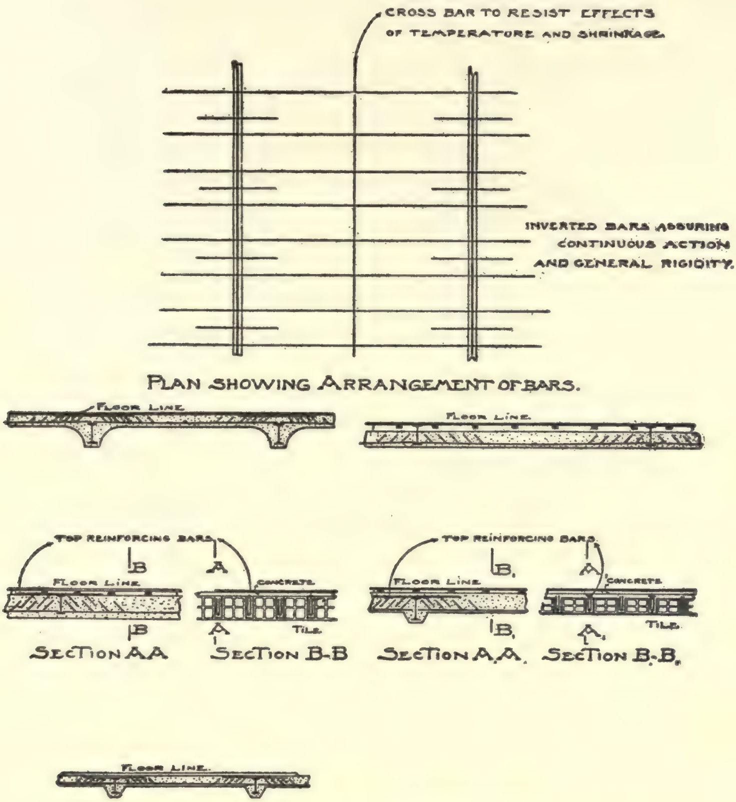


FIG. 72.—Types of reinforced floors.

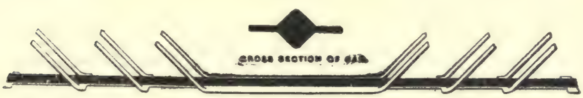


FIG. 73.—Kahn bar.

of the rock was smoothed with concrete but no attempt was made to fill it up to the gradient.

The outer forms were taken off in three days and the inner, in five days. The weather was cool and almost freezing.

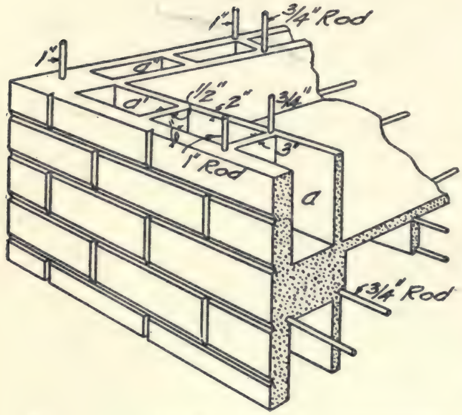


FIG. 74.—Ransome system of reinforced concrete building.

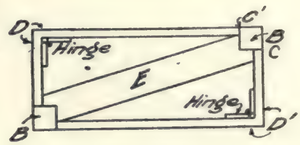


FIG. 75.

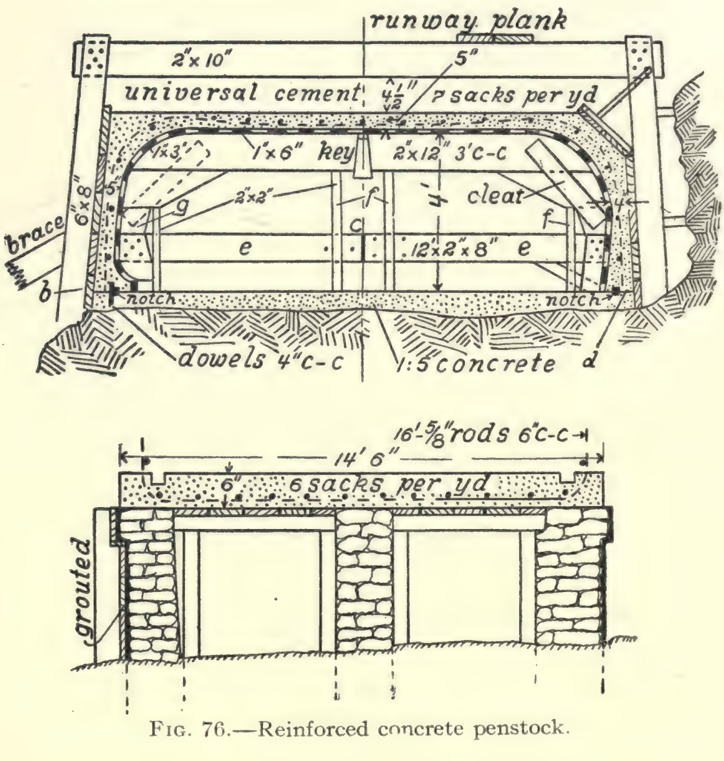


FIG. 76.—Reinforced concrete penstock.

The costs for the first 182 feet were as follows:
Penstock resting on solid rock and as shown in Fig. 76.

ITEMIZED COST PER FT.

Forms, inner, making	$.67
Forms, erecting	.57
Lumber at $30 per thousand	1.08
Steel, placing and hauling	.22
Cement, 5½ sacks	3.00
Dowel pins	.09
Sand, .61 cubic yards at 50c	.30
Labor, mixing, tamping, etc	.83
Concreting rock bottom, labor	.80
Washing inside	.07
	$7.63

COST PER YD.

Concrete, labor, mixing, tamping, etc	$1.36
Concrete, cement, 7 sacks	4.40
Concrete, sand, 1 cubic yard	.50
Forms, making and erecting	2.00
Forms, lumber	1.80
Steel, .90 lb. at $1.83	1.65
Steel, placing	.28
Steel, hauling	.13
	$12.12

Placing steel per lb. (upper) and (lower) .... $.003

The second 182 feet of penstock cost $10.75 per cubic yard. As each 182 foot section is built the cost of the inner forms is decreased. Also, the men become accustomed to the work and do it more cheaply.

Where the penstock rested on piers as shown in the lower part of Fig. 76, the cost was as follows:

Forms, lumber	$0.36
Forms, labor at $2	.27
Concrete, labor	1.14
Concrete, cement, 6 sacks	3.30
Sand	.50
Steel, placing	.27
Steel, 175 lb. at $1.83	3.20
	$9.04 per yard.

In the above work the bottom was difficult to get to, hence the high cost of labor on concrete.

Where the penstock was carried over piers, the cost per foot was as follows:

Part above bottom	$5.80
Bottom	2.17
Piers 18 in. thick and average height 30 in.	1.00
	$8.97 per foot.

The intake at the inlet of penstock contained 59 yards concrete and 5,000 pounds steel. The form work was quite difficult. consisting of floor, beams and 10 in. walls.

The cost of concrete was as follows:

Forms, labor	$1.00
Forms, lumber	0.00
Concrete, labor	.78
Concrete, cement, 7 sacks	3.85
Concrete, sand	.50
Concrete, washing and trimming	.15
Steel, .85 lbs, at $1.83	1.56
Steel, placing	.26

A reinforced concrete abutment 140 feet long and about 16 feet high cost per yard, as follows:

Forms, labor	.96
Forms, removing and trimming	.23
Forms, lumber (old)	.00
Steel, 45 lbs	.82
Steel, placing	.10
Cement	3.30
	$5.45 per yard.

The power house was entirely of reinforced concrete and built for three 35 in. turbines under a head of 13 feet. Each turbine is in a separate wheel pit. The walls are mostly 10 in. and 16 in. thick.

The cost of the concrete in the power house was as follows:

Forms, labor	$2.12
Forms, lumber	2.00
Pump	.40
Concrete, labor	2.00
Concrete, cement	3.30
Sand	.50
Washing and trimming concrete	.10
Steel, at $183	1.10
Steel, placing	.50
	$12.02 per yard.

The forms in all cases were carried up full height of the structure before the concrete was placed. While this costs more for lumber work is much more rapid. The above power house was built in three weeks. and in winter weather.

BUILDING BLOCKS.

There are now on the market many patent moulds for making concrete building blocks, but there is no reason why the engineer should not make his own moulds, as it frequently happens that it is desired to make many special blocks.

In Fig. 77 is shown a mould which is easily made, and which gives good results and a cheap block. The face board *A* has fillets *D* nailed to it to from the joints as shown. The sides *E* are set up and the spacing boards *F* slipped into the grooves formed

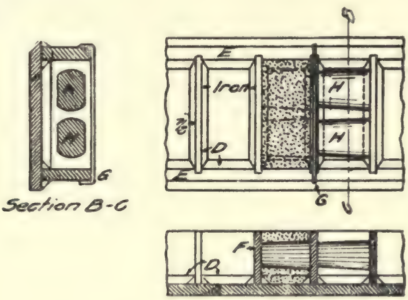


FIG. 77.

by the fillets on the facing boards and the corresponding slits in the side boards. The face of the facing boards is lined perfectly smooth with metal before the fillets are nailed on. All the wood which comes in contact with the concrete must be thoroughly soaked in oil to prevent warping and sticking to the concrete.

Both sides of the spacing boards should also be lined with iron. When the mould is all assembled the clamps *G* are put on, then mortar 1 to 2 is poured in so that it stands at the top edge of the fillets. The concrete (mixed rather damp) is then filled in so that it comes about up to the centre of the cores *H*. The cores are then pressed down between the spacing boards and into the concrete and the mould then filled up over and around the cores. The cores should be covered with sheet metal or

made entirely of it. These are put in the mould by guess, a half-inch one way or the other making no difference. The facing plank should be two or three inches thick, and can be from 12 to 14 or 16 feet long. When filled, the mould should be left for at least four days, when the clamps are knocked off, the blocks taken out and the cores driven out. The cores should have a slight taper. One of the good points of this mould is that the face of the block is at the bottom of the mould, so that it is an easy matter to get a fine, smooth face on the block. Where the face is formed in a vertical position there is sure to be a difference in the hardness and color of the top and bottom edges. The moulds, while drying, must be sprinkled twice a day and kept shaded from the sun. A 1-3-5 mixture makes a strong enough block for all ordinary purposes.

If desired, projections may be put on the spacing boards so that there will be cavities in the blocks to fill with mortar while laying, thus binding the whole wall more strongly together.

The actual cost of these blocks is from 10 to 12 cents per superficial square foot. A wall built of building blocks and nine inches thick equals a brick wall 13 inches thick. To lay a nine-inch wall which would have the same superficial area as a 13-inch brick wall, using 1000 bricks, would cost $7.

By adding about ½ to 3 per cent. of red iron oxide by weight to the cement sand (1 to 1 mixture), the concrete blocks may be made to represent sandstone. Ultra-marine blue added in the same proportion produces a slate or bluish limestone effect. The strength of the concrete is slightly increased by the coloring matter. Ultra-marine green and vermilion can also be used.

Bus hammering the surface of the concrete gives it a fine appearance, and only costs 1½ to 2 cents per square foot. A wall of blocks will require one-fourth the mortar and one-third the labor that a 13-inch brick wall will.

A brick wall 13 inches thick, faced with pressed brick, will cost, per superficial square foot, as follows :

7 pressed brick @ $30 per 1000.......	$0.02	per square foot
14 common brick @ $10 per 1000.......	.14	" "
Mortar and Labor....................	.30	" "
Total.........................	.65	" "

The same wall, built of building blocks would cost from 15 to to 20 cents per square foot. It takes one-third cubic yard of mortar to lay 100 blocks 24x8x8 inches, and a mason should lay ten blocks per hour. Labor and mortar costs four to five cents per superficial square foot.

Strength of Materials.

It is the purpose of the author to here give a clear and concise treatment of the subject without going into laborious explanations and complicated reasoning. The engineer wants the strength of a beam or column and wants it quick. It does not matter if the result is not *exact* so long as it gives a *reasonable* degree of accuracy.

DEFINITIONS.

A *moment* at a given point is the product of a force and the distance between the given point and the point of application.

The *neutral line* is a line which passes through that part of the section in which there is no strain, neither compression nor tension.

The *moment of inertia* of a section about a certain axis is the sum of the products of the elementary particles of the section and the square of their distance from that axis.

The *moment of resistance* of a section is the quotient obtained by dividing the moment of inertia by the distance of the outside fibers (in which the strain is a maximum) from the neutral line.

The *radius of gyration* of a section is the distance from the axis at which the sections if concentrated would have the same moment of inertia as before.

SYMBOLS.

P = *Concentrated* safe load at any point.

p = Safe pressure per square inch of area for columns.

A = Area of section in square inches.

F = Factor of safety.

W = Load, *uniformly* distributed, in pounds = total safe load + weight of beam.

L = Length of clear span in inches.

l = Length of column in inches.

M = Bending moment, in inch pounds, any section.

d = Depth or height of section from out to out, in inches.

n = Distance of center of gravity of section, from top or from bottom in inches.

S = Safe stress per square inch in extreme fibers of beam either top or bottom, in pounds according as n relates to distance from top or from bottom of section = safe strength.

D = Maximum deflection, in inches.

I = Moment of inertia of the section, neutral axis through the center of the section.

R = Section modulus = moment of resistance. (Given in tables for standard sections.)

r = Radius of gyration, in inches.

E = Modlus of elasticity.

GENERAL FORMULAS.

Beams:

$$\frac{M}{R} = S, \ R = \frac{I}{n}, \ r = \sqrt{\frac{I}{A}}$$

$$M = \frac{S\,I}{n} = S\,R, \ S = \frac{M\,n}{I} = \frac{M}{R}$$

Steel columns in buildings

$$p = 17100 - 57\frac{l}{r}$$

Steel struts in trusses

$$p = 13500 - 50\frac{l}{r}$$

Wrought iron columns

$$p = \frac{9000}{1 + \dfrac{l^2}{36000\,r^2}}$$

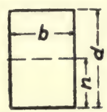
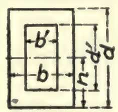
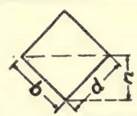
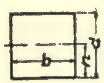
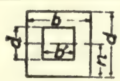
TABLE XXVIII.

STRENGTH OF MATERIALS IN POUNDS PER SQUARE INCH.

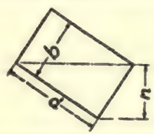
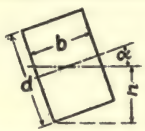
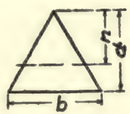
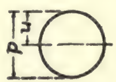
	E Modulus of Elasticity	Ultimate Strength per square inch. (Tension)	Ultimate Strength per square inch. (Single shear) across grain)	Ultimate Strength per square inch. Columns, etc. (Compression)	Ultimate Strength for Beams, Flecture
Ash	1,600,000	16,000		6,800	5,000
Beech	1,300,000	11,500		7,000	5,000
Birch	1,400,000	15,000		8,000	480
Brass, cast	9,200,000	18,000		10,300	
Brass, wire	14,2000,00	49,000			
Cedar				3,500	
Chestnut	1,000,000			4,000	320
Copper, cast	18,000,000	30,000			
Copper, wire	18,000,000	60,000			
Concrete, 6 mos. old		0		700	
Concrete, 1 year old		0		1,000	
Elm	1,000,000				4,200
Glass	8,000,000				
Iron, cast	12,000,000 to 23,000,000	16,000	20,000	80,000 to 100,000	33,000
Iron, cast, average	17,500,000			100,000	
Iron, wrought bars, sheets and plates	18,000,000 to 40,000,000	50,000	47,000	36,000	
Iron, wire	26,000,000	56,000			
Iron, wire ropes	15,000,000				
Lead, sheet	720,000	3,300			
Granite				8,000 to 32,000	
Oak (white)	1,000,000 to 2,000,000	2,000 to 10,000	400 to 700	†4,000 to 7,000	4,000
Oak, average	1,500,000				
Pine, white	1,600,000	7,000	200 to 500	† 750 †3,500	3,200
Pine, yellow	1,600,000	12,000	250 to 600	†1400 †5,000	5,400
Spruce	1,600,000	8,000	200 to 500	† 600 †4,000	3,700
Hemlock		6,000		3,000	3,000
Steel bars	29,000,00 to 42,000,000	45,000 to 120,000		45,000 to 120,000	64,000
Steel, average	35,500,000	100,000	60,000		
Oregon Pine				4,500	
Sycamore	1,000,000				4,000
Brick and cement		280		1,000	
Limestone				2,600 to 18,000	750
Sandstone				2,800 to 16,000	1,000
Best Leather belting		1,000			

† Cross grain.

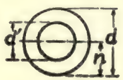
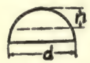
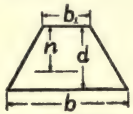
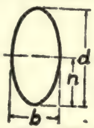
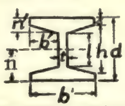
TABLE XXIX.—PROPERTIES OF VARIOUS SECTIONS.

Section.	Area of sect. A	Dis. from axis n	Moment of Inertia I	Section Mod. $R = \frac{I}{n}$	Rad. of Gyration $r = \sqrt{\frac{I}{A}}$
	$b\,d$	$n = \frac{d}{2}$	$\frac{d^3 \times b}{12}$	$\frac{b \times d^2}{6}$	$\frac{d}{\sqrt{12}}$
	$d^2 - d'^2$	$n = \frac{d}{2}$	$\frac{d^4 - d'^4}{12}$	$\frac{d^4 - d'^4}{6\,d}$	$\sqrt{\frac{d^2 + d'^2}{12}}$
	d^2	$n = \frac{d}{\sqrt{2}} = .707\,d$	$\frac{d^4}{12}$	$\frac{d^3}{6\sqrt{2}}$	$\frac{d}{\sqrt{12}} = .298\,d$
	$b\,d$	$n = \frac{d}{2}$	$\frac{b\,d^3}{12}$	$\frac{b\,d^2}{6}$	$\frac{d}{\sqrt{12}}$
	$b\,d - b_1\,d_1$	$n = \frac{d}{2}$	$\frac{b\,d^3 - b_1\,d_1^3}{12}$	$\frac{b\,d^3 - b_1\,d_1^3}{6\,d}$	$\sqrt{\frac{b\,d^3 - b_1\,d_1^3}{12\,(b\,d - b_1\,d_1)}}$

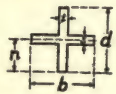
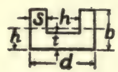
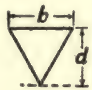
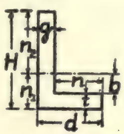
PROPERTIES OF VARIOUS SECTIONS.—*Continued.*

Section.	Area of sect. A	Dis. from axis n	Moment of Inertia I	Section Mod. $R = \frac{I}{n}$	Rad of Gyration $r = \sqrt{\frac{I}{A}}$
	$b\,d$	$n = \frac{b\,d}{\sqrt{b^2+d^2}}$	$\frac{b^3\,d^3}{6\,(b^2+d^2)}$	$\frac{b^2\,d^2}{6\sqrt{b^2+d^2}}$	$\frac{b\,d}{\sqrt{6\,(b^2+d^2)}}$
	$b\,d$	$n = \frac{d\cos\alpha + b\sin\alpha}{2}$	$\frac{bd}{12}\left(\frac{d^2\cos^2\alpha + b^2\sin^2\alpha}{d\cos\alpha + b\sin\alpha}\right)$	$\frac{db}{6}\left(\frac{d^2\cos^2\alpha + b^2\sin^2\alpha}{d\cos\alpha + b\sin\alpha}\right)$	$\sqrt{\frac{d^2\cos^2\alpha + b^2\sin^2\alpha}{12}}$
	$\frac{b\,d}{2}$	$n = \frac{2\,d}{3}$	$\frac{b\,d^3}{36}$	$\frac{b\,d^2}{24}$	$\frac{d}{\sqrt{18}} = .236\,d$
	$\frac{\pi\,d^2}{4}$	$n = \frac{d}{2}$	$\frac{\pi\,d^4}{64}$	$\frac{\pi\,d^3}{32}$	$\frac{d}{4}$

PROPERTIES OF VARIOUS SECTIONS.—*Continued.*

Section	Area of sect. A	Dis. from axis n	Moment of Inertia I	Section Mod. $R = \frac{I}{n}$	Rad. of Gyration $r = \sqrt{\frac{I}{A}}$
	$.785\,(d^2 - d_1^2)$	$n = \frac{d}{2}$	$.049\,(d^4 - d_1^4)$	$.098\,\frac{(d^4 - d_1^4)}{d}$	$\frac{\sqrt{d^2 + d_1^2}}{4}$
	$.393\,d^2$	$n = .288\,d$	$.007\,d^4$	$.024\,d^3$	$.132\,d$
	$\frac{b + b_1}{2} \times d$	$n = \frac{b_1 + 2\,b}{b + b_1} \times \frac{d}{3}$	$\frac{b^2 + 4\,b\,b_1 + b_1^2}{36\,(b + b_1)} \times d^3$	$\frac{d^2 + 4\,b\,b_1 + b_1^2}{12\,(b_1 + 2\,b)} \times d^2$	$\frac{d}{6(b + b_1)}\sqrt{2(b^2 + 4bb_1 + b_1^2)}$
	$\frac{\pi\,b\,d}{4}$	$n = \frac{d}{2}$	$.049\,b\,d^3$	$.098\,b\,d^2$	
	$t\,d + 2\,b'\,(s + n')$	$n = \frac{d}{2}$	$\frac{1}{12}\left[bd^3 - \frac{1}{4g}\,(h^4 - l^4)\right]$	$\frac{2\,I}{d}$	

PROPERTIES OF VARIOUS SECTIONS.—*Continued.*

Section.	Area of sect. A	Dis. from axis n	Moment of Inertia I	Section Mod. $R = \frac{I}{n}$	Rad. of Gyration $r = \sqrt{\frac{I}{A}}$
	$t\,d + s\,(b - t)$	$n = \frac{d}{2}$	$\frac{t\,d^3 + s^3\,(b - t)}{12}$	$\frac{t\,d^3 + s^3\,(b - t)}{6\,d}$	
	$b\,d - h\,(b - t)$	$n = \frac{2\,b^2\,s + h\,t^2}{2\,A}$	$\frac{2\,s\,b^3 + h\,t^3}{3} - A\,n^2$	$\frac{I}{b - n}$	
	$\frac{b\,d}{2}$		$\frac{b\,d^3}{4}$		$\frac{d}{2}$
	$h\,t + H\,s$	$n_1 = \frac{1}{2}\,\frac{S\,H^2 + h\,t^2}{S\,H + h\,t}$ $n_2 = H - n_1$	$\frac{1}{3}\,(d\,n_1^3 - h\,b^3 + s\,n_2^3)$		

BEAMS.

Mr. A. L. Johnson gives the following method for designing reinforced concrete beams:

All beams are considered as being 12 inches wide and as having $e = \frac{h}{10}$. (See Fig. 78.)

For ordinary concrete of 1-3-6 mixture or where 1-2-5 is used but the mixing not of the best, so that the modulus of elasticity, E_c of the concrete = 3,000,000 per square inch, and where E_s the modulus of the steel = 29,000,000 pounds per square inch. Elastic limit of the steel 50,000 pounds per square inch. $f_t =$ 200, $f_c = 2000$.

Then $y_1 = .331\,h$; $\frac{a^2\,b}{d} = .64$ per cent. = percentage of the

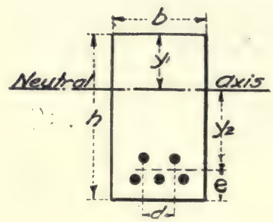


FIG. 78.

whole area of beam which is steel, wherein d = spacing of bars and a = area of one bar.

$M_0 = 3620\,h^2$ = resisting moment = $M\,F$, where F is the factor of safety used, and M_0 = bending moment.

For a better grade of concrete such as would be made of trap rock, good gravel or good limestone in the proportion of 1-2-4, and well made. In this case E, the modulus for the concrete, = 2,400,000. f_c = the compressive strength (breaking) = 2400 pounds per square inch. f_t = tensile strength of the concrete 200 pounds per square inch and the same values for the steel as the above.

Then, $y_1 = .418\, h = \frac{a^2\, b}{d} = .132\, h = 1.1$ per cent.

$$M_0 = 5505\, h^2 = M\, F.$$

In the above, F should be taken at 5 or 6. M = bending moment in inch pounds for beam loaded. (See case 4, page 123.)

EXAMPLE:—A beam 12 inches wide and of 10 foot span sustains a uniform load of 20 feet of water. Find the necessary area of the reinforcing bars: mixture good, and of 1:2:4 concrete.

TABLE XXIXa.

TABLE FOR USE IN DESIGNING REINFORCED CONCRETE BEAMS.

A. L. JOHNSTON. BEAMS 12″ WIDE. 1:2:5 CONCRETE.

M_0	h*	q†	M_0	h	q
50,000	3.00	.397	1,000,000	13.4	1.77
100,000	4.24	.530	1,500,000	16.4	2.17
150,000	5.20	.687	2,000,000	19.0	2.50
200,000	6.00	.793	2,500,000	21.25	2.80
250,000	6.71	.886	3,000,000	23.25	3.00
300,000	7.35	.971	3,500,000	25.10	3.32
350,000	7.94	1.048	4,000,000	27.00	3.55
400,000	8.48	1.120	4,500,000	28.50	3.76
450,000	9.00	1.188	5,000,000	30.00	3.97
500,000	9.48	1.252	5,500,000	31.5	4.16
550,000	9.94	1.313	6,000,000	33.00	4.34
600,000	10.38	1.373	6,500,000	34.25	4.52
650,000	10.81	1.428	7,000,000	35.50	4.69
700,000	11.22	1.482	7,500,000	36.75	4.85
750,000	11.61	1.535	8,000,000	38.00	5.01
800,000	12.00	1.585	8,500,000	39.00	5.17
850,000	12.36	1.633	9,000,000	40.25	5.32
900,000	12.72	1.68	9,500,000	41.5	5.47
950,000	13.07	1.726	10,000,000	42.5	5.60

* h is the depth of beam in inches from top to bottom surface of concrete.

† q is the area of steel in square inches.

The load W = 12,600. Neglecting dead load of beam.

$$\frac{a^2\, b}{d} = .132\, h = 1.1 \text{ per cent.} \quad M = 5505\, h^2 = M\, F.$$

$$M_0 = \frac{5\,(12600 \times 10 \times 12)}{8} = 945{,}000$$

$5505\, h^2 = 945{,}000$, and $h = 13$ inches. The area of the steel then is, 13 inches $\times$ 12 inches $\times$ .011 = 1.71 square inches.

These formulas will serve for any reinforcing which is especially prepared to prevent slipping.*

Tables giving safe loads on floors and beams may be had from the various companies. The Trussed Concrete-Steel Company of Detroit publish a useful booklet.

Columns and Foundations.

Numerous experiments seem to indicate that columns made of reinforced concrete and less than 20 to 25 diameters in height do not fail by flexture (bending) but invariably crush. Therefore under these conditions the column need not be calculated for flexture. The crushing strength of the concrete may be taken at from 1500 pounds per square inch to 1800. Using a factor of safety of 5 to 6 we have the safe strength of 250 to 350 pounds per square inch. Concrete-steel columns wound with wire or hooped may have a safe strength of 800 to 1400 pounds per square inch.

The building ordinances of Chicago allow a maximum load on concrete of 8 to 15 tons per square foot. Trautwine gives the *crushing* strength as follows:

For concrete		1	month	old	12	to	18	tons	per	square	foot.
"	"	6	"	"	48	"	72	"	"	"	"
"	"	12	"	"	74	"	120	"	"	"	"

Kidder gives 14½ tons per square foot as the safe load.

A factor of safety of from 6 to 10 should be used.

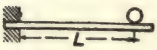
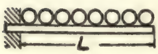
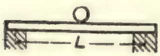
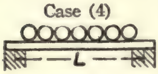
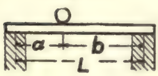
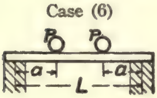
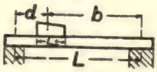
Crushing strength for neat (all cement) cement = 25 to 60 tons per square foot.

*The author has lately been using nothing but plain round mild steel bars which may be purchased for about $0.018 per pound delivered. One and one-fourth per cent. of mild steel is used in this case, and in all cases where rods lap the ends pass each other 40 diameters.

Where high carbon rods are used they stretch long before their full strength is utilized, and the concrete cracks. Therefore the only value of the more expensive steel is lost.

TABLE XXX.

PROPERTIES OF BEAMS.

	Bending Moment M_{max}	Max. Load	Pressure at Support P_s	Deflection D
Case (1) 	$P\,L$	$\frac{S\,R}{L}$	P	$\frac{P\,L^3}{3\,E\,I}$
Case (2) 	$\frac{WL}{2}$	$\frac{S\,R}{2\,L}$	P	$\frac{WL^3}{8\,E\,I}$
Case (3) 	$\frac{P\,L}{4}$	$\frac{4\,S\,R}{L}$	$\frac{P}{2}$	$\frac{P\,L^3}{48\,E\,I}$
Case (4) 	$\frac{WL}{8}$	$\frac{8\,S\,R}{L}$	$\frac{P}{2}$	$\frac{5}{384}\,\frac{WL^3}{E\,I}$
Case (5) 	$\frac{P\,a\,b}{L}$	$\frac{S\,R\,L}{a\,b}$	$\frac{P\,b}{L}$ or $\frac{P\,a}{L}$	$\frac{P\,L^3\,a^2\,b^2}{3\,E\,I\,L^4}$
Case (6) 	$\frac{P\,a\,(L-a)}{L}$	$\frac{2\,S\,R\,L}{a\,(L-a)}$	$\frac{P}{2}$	$\frac{2PL^3a^2(L-a)^2}{3\,E\,I\,L^4}$
Case (7) 		$\frac{L_1\,P_s^2}{2\,RS-P_s\,a}$	$\frac{P\,(2\,b+L_1)}{2\,L}$ or $\frac{P\,(2\,a+L_1)}{2\,L}$	

NOTE.—All lengths are measured in inches, and all forces in pounds. The moments of resistance and inertia will be found under the properties of sections.

TABLE XXXI (Kidder).

Safe Bearing Load for Different Soils in Tons per Square Foot.

	Minimum.	Maximum
Rock, hardest kind....................	200	..
Rock, equal to Ashler masonry.........	25	30
Brick, equal to Ashler masonry.........	15	20
Brick, of poor quality..................	4	7
Clay in thick beds, always dry..........	4	6
Clay in thick beds, moderately dry......	2	4
Clay in thick beds, soft and wet........	1	2
Gravel and coarse sand................	8	10
Sand, fine and compact................	4	6
Sand, fine, clean and dry..............	2	4
Alluvial soils and uncertain sand........	0.5	1

Safe pressures are given by Rankine to be 1 to 1½ tons per square foot on tamped earth; 2 to 3 tons per square foot on compact gravel and dry sand; or 4 to 6 tons per square foot where a few inches settlement may be allowed; 1 to 2½ tons per square foot safe load no pure soft clay; 2 tons per square foot on silty soil will settle 3 to 12 inches in a few years.

TABLE XXXII (Kidder).

Maximum Safe Load in Pounds per Square Inch on Different Kinds of Masonry for Bearing Plates under Columns and Girders.

For granite.................................	1000
" best grades of sandstone......................	700
" soft sandstone............................	400
" hard stone rubble.........................	150 to 250
" extra hard brick in cement mortar..............	150 to 200
" good hard brick (Eastern) in cement mortar....	120
" common brickwork..........................	100
" good Portland cement concrete.................	200
" good Portland cement concrete reinforced........	400

One or more holes through bottom of column bearing plates should be left so that it can be determined whether or not the grout fills up underneath.

FACTORS OF SAFETY.

The factor of safety is that figure by which the ultimate strength is divided in order to get the safe strength, and therefore is the most important of all engineering data. The reckless engineer adopts a low factor of safety trusting to luck for future fame as a close calculator while the conservative engineer selects the higher values. It is, therefore, largely a personal equation.

In Table XXXIII the factors of safety are given as recommended by Unwin, Gordon, etc., and Pencoyd, Carnegie & Cambrian Steel Companies. They are very conservative; judgment must be exercised in their use however. Where the material is exposed to wear or rust a certain amount of the area must be allotted to this loss and the factor applied to the remaining portion. If samples of the materials are frequently tested, a much lower factor may be used based on these tests.

TABLE XXXIII.

FACTORS OF SAFETY.

Name of Material.	Steady Load, No Vibration Dead Load.	Fluctuating Loads. Vibrations.	Shocks as Machine.	Temporary Structure.	Tensile Dead Load.
Steel	5	5 to 7	15	4	8 to 12
Cast Iron	6	15	20	..	Very uncertain
Steel Shafting	5	8	12	4	..
Leather Belts	10	12	14	6	..
Stay Bolts	6	7	12	4	..
Wood Dry	6	12	20	8	10
Wood Green	15	18	30	12	..
Brickwork	15	25	30	14	..
Steel Columns	6	6 to 10	12	3	..
Nickel Steel	5	5 to 7	10	3	8
Bronze	5	6	8	3	7

EXAMPLES.

In nearly all pocket-books there are tables giving the properties of standard sections of structural steel and "Sample's." " Properties of Steel Sections " gives the properties of all sorts

of built-up sections*. It is assumed that the reader possesses such a table in solving the following examples:

Beams.

EXAMPLE 1.—Given a 24-inch I-beam of 16-foot span weighing 100 pounds per foot: what is the safe uniform load it will sustain? From the tables for standard section, I is 2380; n, the distance of the neutral axis, $A\ A$ from the extreme fibers is 12 inches, therefore (Fig. 79)

$$R = \frac{I}{n} = \frac{2380}{12} = 198.3$$

R is given in the tables in the column headed Section Modulus. From case (4),

$$\text{Load} = \frac{8\,S\,R}{L}$$

where 16,000 = safe strength of the steel.

$$W = \frac{2\,S\,R}{3\,L} = \frac{2 \times 16000 \times 198.3}{3 \times 16} = 132{,}200 \text{ lbs.}$$

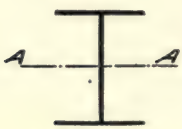


FIG. 79.

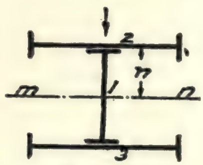


FIG. 80.

The difference between $\frac{8\,S\,R}{L}$ and $\frac{2\,S\,R}{3\,L}$ is due to the fact that the equations in properties of sections gives inch pound values thus $\frac{8\,S\,R}{12\,L} = \frac{2\,S\,R}{3\,L}$.

If the beam is built up as shown in Fig. 80, and we have no tables giving the value of I or R, I may be found as follows:

The moment of inertia of any built-up section about an axis is equal to the sum of the moments of inertia of those sections through whose centers of gravity the axis may pass, plus the sum of the moments of all sections through whose centers of gravity the axis does not pass, plus the area of all such sections multiplied by the square of their distances from the axis.

*With permission of the publishers several of Sample's tables are given in Chapter IX.

EXAMPLE 2.—Find I for Fig. (80), first about the axis $m\ n$, which passes through the center of gravity of I-beam (1), and is at the distance n from the center of gravity of (2) and (3), then $I = I$ for beam $(1) + 2$ [area of beam (2) or (3) $\times n^2$] $+ 2$ (moment of inertia of beam 2 or 3).

Figured on this axis, the load should be in the direction of the arrow, because the neutral axis passes at the point where there is neither tension nor compression.

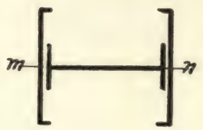


FIG. 81.

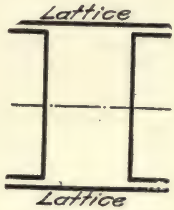


FIG. 82.

In the case of the axis passing through the center of gravity of all the members as in Fig. (81), $I =$ [moment of inertia of the I-beam] $+ 2$ [moment of inertia of one of the two channel beams.]

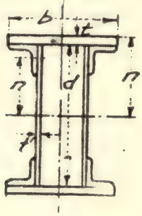


FIG. 83.

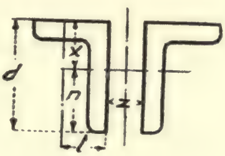


FIG. 84.

Care must be exercised in selecting I for the various sections, as in the tables I is given for both axes.

In the case of a latticed beam, Fig. 82, the lattice is not considered. Thus I = twice the I for one channel.

For Fig. 83

$$I = 2\left[\frac{t\,b^3}{12} + b\,t\,n'^2\right] + 4\left[(\text{area of angle} \times n^2) + (I \text{ of angle})\right]$$

$$+ \left[\frac{d^3 \times 2\,t}{12}\right]$$

The distance X, Fig. 84, is found by deducting the values

given in tables for standard sections for the perpendicular distance from center of gravity to back of flanges, from the distance y.

I for two angles (Fig. 84) = twice I for a single angle.

EXAMPLE 3.—Find the safe center load, P, which two 4x4x½-inch angles will support. Span = 6 feet, Z = 1 inch.

From case (3), $P = \dfrac{RS}{3L}$

From tables for standard sections, I on axis 1, 1, = 5.56.
Therefore I for the two angles = 11.12.

$$R = \frac{I}{n}$$

In the tables x is given for the above angle, as 1.18 $\therefore n = d - x = 4 - 1.18 = 2.82$

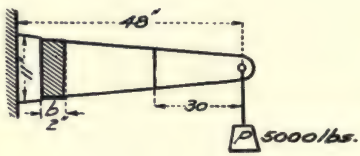


FIG. 85 .

$$R = \frac{11.12}{2.82} = 3.94 \quad \text{and } P = \frac{3.94 \times 16000}{3 \times 6} = 3502$$

EXAMPLE 4.—Design the beam shown in Fig. 85 of rectangular section made of cast iron. To start with assume a thickness b of 2 inches. From Table XXVIII the ultimate strength of cast iron in flexture, is 36,000. The load is to be a steady stress, therefore the factor of safety is 6 and the safe strength is 6,000 pounds.

Now $M = S\,R$ or, $W\,L = 6000 \dfrac{b\,d^2}{6}$ and substituting the proper values, $5000 \times 48 = 6000 \dfrac{2 \times d^2}{6} = 120$, and $d = 10.95$ inches; call this 11 inches.

To get the depth at any other point, say 30 inches from the end, proceed as before, only substituting 30 inches instead of 48 inches.

In the design of shafting we frequently have the condition shown in Fig. 86, and have to find the maximum bending moment M. Suppose the gear or pully weighs 1111 pounds, and the belt chain or tooth pull tending to rotate it, regardless of I or *r. p. m.* = 4111 pounds, then

$$M = \frac{(4111+1111)\times(6\times12)\times(3\times12)}{(3\times12)+(6\times12)} = 125{,}330 = SR = S\ .098d^3$$

S for steel is about 10,000. Substituting and solving for d we have a shaft about $5\frac{1}{4}$ inches in diameter.

If the wheel is at the center of the span.

$$M = \frac{(4111+1111)\times(3+6)\times12}{4} = \frac{PL}{4} \text{ in inch pounds.}$$

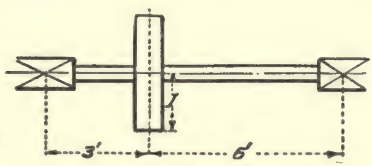


FIG. 86.

EXAMPLE 5.—A standard I beam, 20 feet span, is loaded at the center with a load P, of 10,000 pounds. Find proper size of beam:

From case (3) $R = \frac{3PL}{S}$

Therefore

$$R = \frac{3\times10{,}000\times20}{16{,}000} = 37.5$$

From tables for standard steel I-beams we find that the section having a section modulus of 37.5 is a 12-inch-35 pound beam.

Columns.

The radius of gyration $r = \sqrt{\frac{I}{A}}$ plays an important part in the design of columns.

If the column is a built-up section I, the moment of inertia, is found in the same way as for beams.

EXAMPLE 1.—Find r about axis 11, for column (Fig. 87) composed of two latticed 12-inch channels weighing 40 pounds per foot each, and placed six inches apart.

$$I = 2[(\text{area of each channel} \times \text{by } X^2) + (I \text{ for each beam})]$$

From tables for standard channels we find, x for axis, 11, and a 12 inch-40-pound channel to be .722, therefore $X = .722 + 3$ inch, or 3.722 inches: area of channel = 11.76. I for one one channel = 6.63.

Substituting,

$$I = 2\ (11.76 \times 3.722 + 6.63) = 339$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{339}{2 \times 11.76}} = 1.2$$

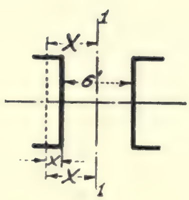


FIG. 87.

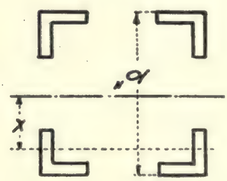


FIG. 88.

Example: The column shown in Fig. 88, is built up of four 3x3x¼-inch angles. Find r

$$I = 4[(\text{area of one angle} \times x^2) + (I \text{ for one angle})]$$

From tables for standard shapes $x = .84$.:

$X = 5 - .84 = 4.16$.

Area of one angle $= 1.44$. I for one angle $= 1.24$.

$$I = 4(1.44 \times \overline{4.16}^2 + 1.24) = 104.6 \qquad r = \sqrt{\frac{104.6}{4 \times 1.44}} = 4.26$$

DESIGN OF MACHINE ELEMENTS.

THE SCREW.

F = force in pounds applied at circumference of hand wheel A at a point B, R inches from center line of screw. P = the distance between threads in inches = pitch of screw = distance

which one complete revolution of hand wheel will raise screw.

W = weight lifted.

Then $$F = \frac{W P}{6.283\ R}$$

where 6.283 = a constant.

EXAMPLE.—What pull must be applied at B to just raise 5000 pounds hanging to the screw? The screw itself weighs 111 pounds and has a pitch of one inch. R = 24 inches. f = .2

Then

Weight of screw........................	111
Friction due to screw, $111 \times .2$............	22.2
Weight lifted............................	5000.0
Friction due to the weight, $5000 \times .2$......	1000.0
W = ...	6133.2

And $F = \dfrac{6\,133.2 \times 1}{6.283 \times 24} = 40.7$ pounds.

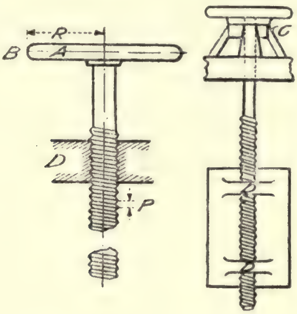


FIG. 89 and 90.

A man turning the wheel A would exert an equal pressure on each side and in the above case would exert 20.35 pounds pressure with each hand.

The *work* of lifting is measured in foot pounds and in the above case would $= 40.7 \times 2\ \pi\ \dfrac{R}{12} \times N$ where $2\ \pi\ \dfrac{R}{12}$ is the circumference of the hand wheel in *feet* and N the revolutions.

A man can, for a minute or two, perform work equal to 17280 foot pounds per minute. In the above example one revolution $= 40.7 \times 12.566 = 511.4$ foot pounds, and the number of revolutions a man could turn the wheel per minute $= \dfrac{17280}{511.4} = 33.8.$

In 33.8 revolutions the screw would lift the weight $33.8 \times P$ or in the above case 2.82 feet.

Often the screw itself does not move vertically, but is supported on a collar C, Fig. 90, the nut attached to a weight traveling along the screw and lifting the weight W. In this case the friction of the collar is added to that of the nut D.

Knowing the pitch of the screw and the weight it is to sustain, it is a simple matter to design it for strength. From the table giving the shearing strength of materials the safe strength for the material in the threads is found and sufficient area is provided so that the threads will not strip. If it is six inches in diameter at the root of the thread and the thread is one inch thick (see Fig. 91), the safe strength of the thread making one turn around the shaft would be $6 \times \pi \times 1$ inch $\times S$ where $S =$

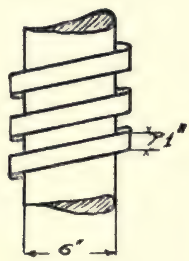


Fig. 91

safe strength of the metal, The same reasoning would apply to the nut.

WINCHES.

Heavy winches usually employ the worm and gear, Fig. 92 for at least one power increase.

L is the distance in inches, a tooth on worm gear moves in one revolution of screw. For a one threaded worm $L = P$ for a two threaded worm $L = 2\,P$, three threaded $L = 3\,P$, therefore, for a two threaded worm and a gear of 50 teeth the shaft A will make one complete revolution for every 25 of the crank C; or disregarding friction one pound exerted at C should lift 25 at B a distance $2\,P$. If the worm is single threaded the ratio would be 1 to 50 and the distance moved P.

Efficiency of a worm gear is about 40 per cent. for *starting*

loads; 50 per cent. for pitch line velocity of 10 feet per minute; 80 per cent. for velocities of 100 feet.

Therefore, in the above case of 50 teeth in gear and single threaded worm of $\frac{1}{2}$ pitch, P, $R = 7$ inches, we would have the force F, applied at C, necessary to start the worm against a resistance at the pitch line at B, of 2000 pounds,

$$F = \frac{2000 \times \frac{1}{2} \times 1.6}{6.243 \times 7} = 36.6 \text{ pounds.}$$

1.6 is gotten from the efficiency being 40 per cent.

36.6 pounds, acting at the 7-inch radius, travels 3.665 feet per revolution. It requires 48 revolutions to raise the gate 24 inches, therefore if the gate is to be raised in one minute, $36.6 \times 3.665 \times 48 = 6432$ foot pounds per minute are required, which is less than half the power a man can exert for short periods.

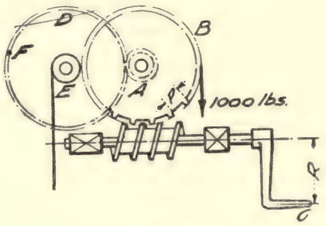


FIG. 92.

For steady work during several hours, a man can only exert about 3500 foot pounds per minute.

Now if the rope lifting the weight is run over a smaller wheel, A, say $\frac{1}{4}$ the diameter of B, the leverage will be increased in the proportion of $\frac{B}{A}$; but the time of lifting is also increased in the same proportion; 36.6 pounds at C will now lift 8000 pounds at A. If a pinion is placed at A and a spur gear D is used, the lifting power on the pitch circle F will be increased in proportion to the ratio of the gears. Suppose this ratio is 1 to 4 in the above axample, then the 36.6 pounds exerted at C will lift 2000×4 or 8000 pounds at F. The efficiency of a pair of well-cut gears should be 97 per cent., and that of uncut and poorly designed gear 90 per cent. Therefore the power delivered at A, in the above example $= 2000 \times \frac{A}{B} \times$ efficiency of gears, or if

the pitch diameter of $A = \frac{1}{2}$ that of B, and the diameter of E is $\frac{1}{2}$ that of D, 8000 pounds will be transmitted at a loss of 3 to 10 per cent; if 10 per cent is lost 7,200 pounds will be the actual weight lifted at F, and so on through any number of reductions.

Force is increased at the *expense* of *motion*, when the energy remains constant.

HOISTING BY ROPE OVER A DRUM.

S = stress in rope in pounds at A Fig. 93. When the weight W is suddenly lifted, owing to slack in the rope, the stress is greatly increased.

R = weight of the whole rope in pounds.

F = equivalent friction in pounds = weight of all moving parts x by f.

f = coefficient of friction.

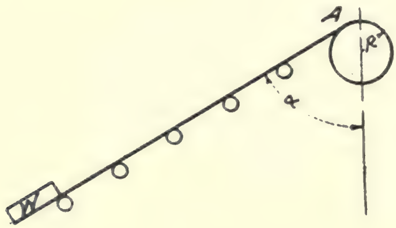


FIG. 93.

Then $S = 2W + R + F$.

W = weight lifted.

f = .01 for vertical hoisting and .02 to .04 for inclined.

EXAMPLE: Required size of rope to hoist vertically, 5,000 pounds through 1500 feet; $S = [(5000 \times 2) + (2 \times 1500)].01\ (5,000 + 3,000) = 13080$ pounds.

If the factor of safety = 7 select rope having ultimate strength of 100,000. (See table LXIII).

If the hoisting is done on an incline, $S = (2W + R) \sin \alpha + F$ and $F = f(W + R)$ cosine α. See Fig. 93.

As the rope passes over the drum the strain is greatly increased, due to the bending of the fibres. If the drum is 45 times the diameter of rope the bending strain = 8/9 of the whole strain on rope and leaves 1/9 of the ultimate load available. If 80 times the diameter of rope the bending strain will have 1/5 of

the ultimate strength of the rope available for use. The bending stress on different ropes are given in the tables.

STEEL CABLES.

$E = 28{,}500{,}000$ = modulus of elasticity.

A = net area of steel. R = radius of drum or sheave.

d = diameter of each wire in the rope.

The net working stress for which the rope is safe will then be the difference in S, found as above, and the safe strength given in Table LXIII for the particular rope.

EXAMPLE: Find safe load for a one-inch cast steel rope running over a six-foot sheave. From Table LXIV the bending stress = 9937 pounds and from Table LXIII the maximum safe stress for a one-inch rope = 22667 pounds. The difference, 12730 pounds, is the safe working load.

The stress due to weight of rope and weight lifted as found by the first formula for S must not exceed this safe load

PULLEY OR GEAR.

When we have a pulley or gear transmitting power we usually wish to know the tension on the belt or rope, in case of a pulley or the pressure on a tooth of a gear.

EXAMPLE:—A pulley 38 inches in diameter runs at 116 revolutions per minute, and transmits 110 horse power. What is the pull at the rim?

$$\text{Pull} = \frac{110 \times 33{,}000}{38 \times .2618 \times 116} = 3144 \text{ pounds}$$

$$.2618 = \text{a constant.}$$

If a gear has the same diameter at the pitch circle the pressure would be the same and would be considered as all acting on one tooth.

CENTRIFUGAL FORCE IN WHEELS.

Let W = weight in pounds of entire rim of pully, gear or fly wheel. R = radius in feet to center of gravity of rim section. S_r = revolutions per minute.

S = total strain on the cross section of rim in pounds. Then $S = .00005427\ W \times R \times S_r^2$.

S, divided by the area of the section gives the strain on the metal per square inch.

SHAFTING.

Consider one end of the shaft, A Fig. 94, fast and on the other end a lever l inches long, with a weight of W pounds. Suppose, for example, we give the shaft a safe strength S of 10,000 pounds

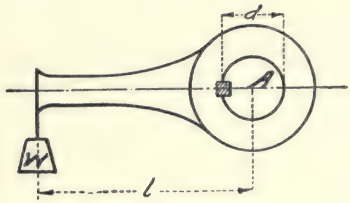


FIG. 94.

per square inch, then the formula for the safe diameter under that stress is $W\,l = .196d^3 \times 10{,}000$; from this d may be found, or having d, S may be obtained.

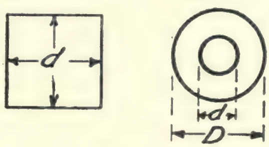


FIG. 95.

For square shafts (Fig. 95) $W\,l = .28d^3S$

For hollow shafts (Fig. 95) $W\,l = \dfrac{A\,D^2 - ad^2}{A\,D}$

1 = area of circle of diameter D.

a = area of circle of diameter d

For maximum *bending* moment see Table XXX.

CHAPTER V.

HYDRAULIC CONSTRUCTION

PILING

No feature of engineering work is more disappointing and at the same time more important than piling. There is always more or less uncertainty as to the cost before beginning, and then as to the efficiency of the job when completed. Many

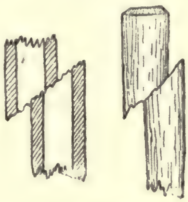


FIG. 96

times piles will appear to drive all right, when in reality they are being sheared off, as in Fig. 96. Sheet piles may look well at the top, when, in fact, there are large holes between them further down.

There are two patent types of wooden sheet piling, the Wake-

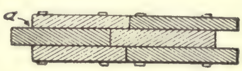


FIG. 97.

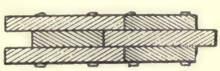


FIG. 98

field (Fig. 97), and the Beardsley (Fig. 98). The Wakefield consists of three planks all of the same width, while the Beardsley pile is composed of two widths.

Each pile has its advantages for certain conditions. The planks for the Beardsley pile being in two widths are sawed out of the log to better advantage than if all of the same width.

Where the pounding will not be too heavy, the planks may be spiked together with 60d wire spikes, and clinched, but generally four to eight-five-eighth-inch carriage bolts are used. Of course, the groove of one pile must be slightly wider than the tongue of the next, otherwise it will bind in driving. The usual method in the case of the Wakefield pile is to place a three-sixteenths-inch shim at *a*. The narrowest planks

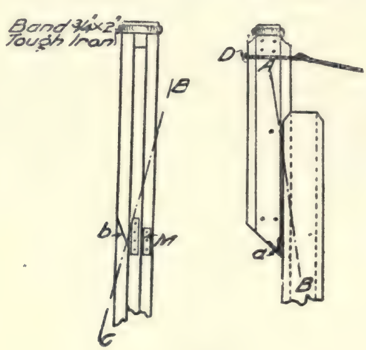


FIG. 99.

in the Beardsley pile are all sawed three-sixteenths-inch thicker than the wide plank to get the desired result.

It requires great experience and a good head to successfully drive sheet piling. Experience with round piling alone is worse than none, as far as sheet piling is concerned. Each pile must

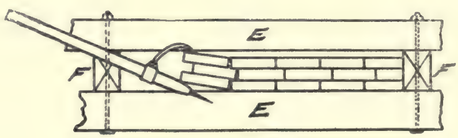


FIG. 100

be given just the proper edge bevel, as at *a* (Fig. 99), or side bevel as at *b*. The edge bevel causes the pile to hug to the preceding one, but if too much is given the piling may get to running in the direction of the line *A B*. The side bevel *b*, is given where the previous pile has run, as shown by the line *B C*. If the bevel is too great the foot of the pile may run out of the groove. The top of the pile is held securely against

the preceding pile by the rope D, which passes back to the steam wench, in the case of a steam driver, or to a pair of double blocks in the case of a horse driver. Too great a pull on this rope will throw the foot of the pile out.

If the pile runs, as shown in Fig. 100, one or more peavies are used to bring it back into line. Timbers E, E are laid along the line and bolted together with spacers, F, F between, to aid in keeping the piling straight, as the piling approaches the spacers, a bolt is put through the timbers and the pile nearest the spacer and the spacer removed. One-inch bolts should be used with heavy washers, so that the timbers may be brought back into place if they have spread any.

It is often necessary to put iron points on the piling, as at M, Fig. 99. These may consist of iron 2x4-inch, bent to fit the bevel. The common dimensions of the plank are 2x12 inches and

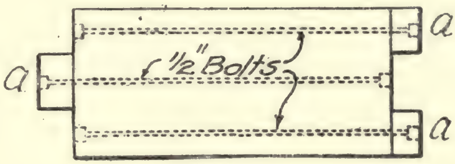


Fig. 101.

2x8 inches, and the wood used may be beach, oak, Southern pine, gum, hard maple, cypress, elm, or any close-grained hard wood. The head of the pile should be banded with a band of the best Norway iron ¾x3 inches, and several bands should be used, so that two or three banded piles will always be ready for use. Where the piling is quite long, say 25 to 40 feet, it must be much heavier, and is made as in Fig. 101. Strips (a) are bolted to the pile with ½-inch carriage bolts to form the tongue and groove.

PILE DRIVERS.

The smaller drivers are usually operated by horse-power, but for large, quick work a steam driver is used, the pile driver being similar to the horse power, except that the engine is set in the frames. The most rapid and satisfactory pile driver is the steam-head driver. The parts work between the leaders the same as a drop hammer, but rests continually on top of the pile. The piston and weight are caused to reciprocate by

steam acting on a piston in the cylinder, at the rate of 80 strokes or less per minute. The rapidity of hitting may be nicely regulated. Piling may be driven in quicksand, hard pan, etc., with this driver, where it would be impossible with the slower type. In quicksand the pile is buoyed up by an amount equal to the weight displaced, and where the blows are few and far between the pile rises between strokes; but with the steam-hammer the weight is on the pile at all times and blow follows blow in such rapid succession that the displaced particles of sand and water do not have time to settle back into place. Also much cheaper grades of wood may be used for the piling, as the splintering effect is less.

A common horse-power pile driver with a 2000-pound drop hammer costs all complete from $150 to $200. The same outfit, with a suitable boiler and engine, will cost from $800 to $1,000. A steam-head driver of the same comparative size will cost from $1,500 to $3,000.

COST OF DRIVING PILING.

TABLE XXXIV.

Cost of Driving Round Wood Piling and Number Driven Per Day.

Horse Power.		Steam.		Steam-Hammer.		Depth in Compact Clay. (Feet)	Depth in Strong Gravel. (Feet)	Depth in Wet Sand. (Feet)
No. of Piles	Cost per Pile	No. of Piles	Cost per Pile	No. per day	Cost			
6 to 12	$1.75	8 to 10	$3.50	15 to 30	$1.50 to $2.		20	
		12 to 14	$5 to $6				25 to 35	
4 to 6	$2 to $2.50	6 to 8	$4 to $6	20 to 50	$1 to $1.50			14
	$2.50 to $3					12		
8 to 10	$2.50						13	
	$2.75*						16	

* Labor was high.

The above costs do not include the pile itself, and represent costs actually attained on various jobs.

The cost of driving sheet piling is about the same per foot. measured across the stream, as that of round piling. It will cost about $1.50 per foot width to drive 12 to 16-foot piles, when the driving is easy, and from $2.50 to $3.00 where it is hard.

The lowest average bid for driving round piling on a large pile dam by several reliable contractors was 45 cents per foot of pile. This included the pile and the sawing off under water. The current was strong and driving average. The piles cost from 10 to 15 cents per foot. In charging so much per linear foot (the usual method) the pile is measured from its point to the sawed-off head, the penetration not being measured, though it, of course, influences the price.

Ordinary round piles for hydraulic work should not cost more than 8 to 15 cents per linear foot for driving.

METHOD OF DRIVING.

Fig. 104 shows a few of the common pile points for round piles. The first is a soft iron strap and the second and third are of cast iron.

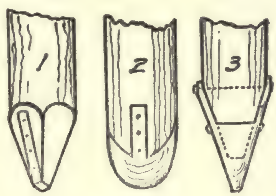


FIG. 104.

The author has found that for soft soils such as sand, silt, and soft clay, the piles drive much better if not pointed at all, but left with a square end; in fact, experience with the round piling indicates that the large end should be the down end in such soils.

Sand is somewhat like a liquid, and has a buoyant force, which will always act to force the pile out of the ground, and depends entirely upon the volume of the pile submerged. When the pile is driven point first this force lifts it out as fast as it can be driven, but when driven butt first this force acts in the same way, but is opposed by a great frictional resistance which would have to be overcome before the pile could be removed.

This resistance represents the work which would have to be done in displacing the shaded volume of sand (Fig. 105.)

Western rivers, such as the Platt and Elkhorn, demonstrated that it was far better practice to drive the piles with the butt-end

down. Of course in this case the small end must be of good proportions so as not to break or broom under the hammer. With this way the pile does not spring back, and, instead of the " flare " hitting towards the sand, it drives away from it.

When the pile driver is mounted on a scow, scattered round piles may be driven twice as rapidly as on land, but when the piling is along the edge of a platform or mat, the cost is less for the land driver.

A heavy hammer and short fall is the most satisfactory, as the pile will be shattered less and more blows can be given. This is especially true for quicksand.

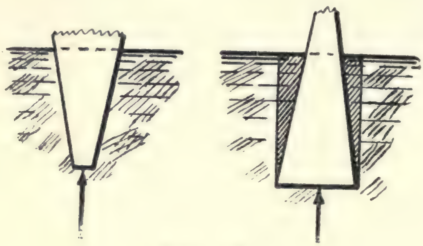


FIG. 105.

Trautwine gives the adhesive power of ice to a pile as 30 to 40 pounds per square inch of surface.

The friction of a metallic pile is about three-tenths that of wood

Jet Driving.

The most successful way to sink round or sheet piling in wet sand is by means of a jet of steam or water. The author had an experience with the Elkhorn River in Nebraska, which thoroughly demonstrated the value of the jet. After vainly trying to drive sheet piles in the sand of the river bed for two months, with an ordinary pile driver, an Edson pile sinking outfit, costing about $150 was used. This is shown in Fig. 106. Two men handled the 1-inch tube, which was constantly moved about so as to loosen up the sand under and around the pile. Two men worked the pump and two men guided the pile. The pile was handled in the leaders of a pile driver, and a 2,000-pound hammer left resting on the head of the pile. When the pile stuck, a few blows of the hammer started it again. With this

simple outfit an average of from 20 to 30 piles, 14 feet long by 12 inches in diameter, were driven per day, at a cost of $1.50 per pile.

Steam can be used in the place of the water. Jetted piles may be of the soft wood. With the jet, round piles averaging 14 inches in diameter can be sunk by means of a strong jet at the rate of a foot per second. Large cylinders may be rapidly sunk in the worst sands, and it seems very strange that this

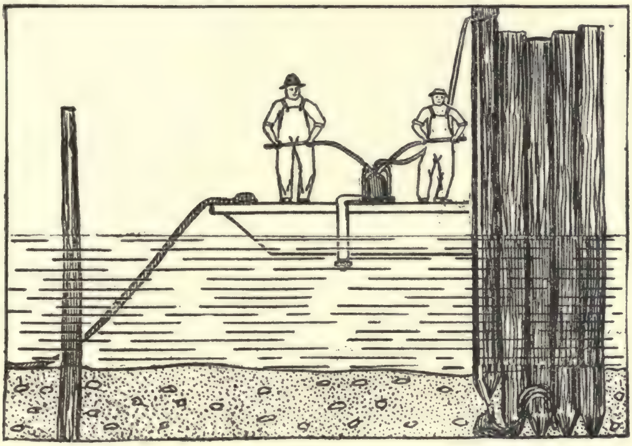


FIG. 106.

method is not more generally adopted. Of course, the jet can only be used for sands, or other soft soils.

CONCRETE PILES.

Wooden piles rapidly decay unless entirely submerged in water. If exposed to sea water they are eaten up by insects. Concrete piles are permanent, and it is only a question of a few years when wooden piles will be a curiosity, having been entirely displaced by the concrete.

There are several patent concrete piles, a noteworthy one being the Raymond pile. This is made by sinking a thin casing of metal the size of the pile and filling it with concrete. Fig. 107 shows the adaptation of the concrete pile, and how eight concrete piles displaces 22 wooden piles. Reinforcing bars may be placed in the molds before filling.

Fig. 108 illustrates a concrete-steel pile, which is made before placing it in the ground. As here shown it is reinforced by three ¾-inch Kahn bars, making 160 pounds of steel for a 20-foot pile. At the bottom end of the pile the three rods converge to

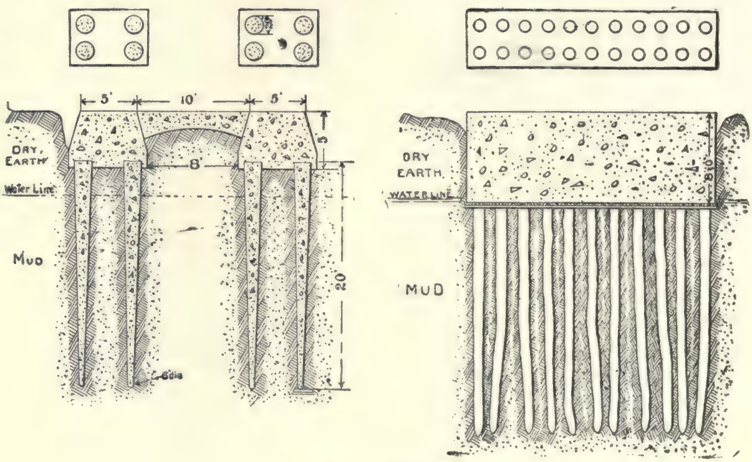


FIG. 107.

a point, and are welded together. The concrete used is made of high-grade Portland cement and clean river gravel, in the proportion of 1 to 3.

The method of constructing the piles is as follows: The molds or forms are set up on one corner, and the concrete placed

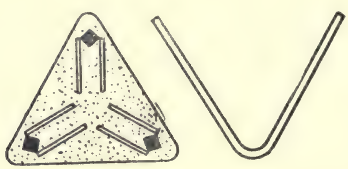


FIG. 108.

in them. Piles can be built in this way in lengths varying from 16 to 26 feet, as required. When molded complete the pile is left to set for about a day, without water, and then kept in the form for a week longer, with continual sprinkling. By that time the concrete has hardened sufficiently to allow the

piles to be lifted out of the form and set away for another period of a week or more, during which time they are kept constantly wet. After this they can be removed for transportation or storage.

The piles can be driven by steam hammers weighing as much as 5000 pounds, with a fall of about 5 feet. The head of the pile is protected from damage in driving by a cushion cap made of alternate layers of iron plate, wood and lead, which is clamped to the pile head. This cap also serves to guide the pile in the leads. Such a pile 20 inches across corners at the top and 6 inches at the bottom would cost about $9.00 all complete, and give the bearing power of four wooden piles.

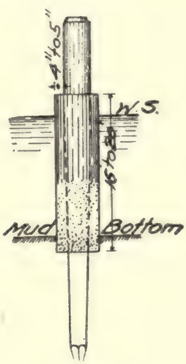


FIG. 109

Protection of wooden piling from the Teredo can be secured by grouting a concrete jacket around the pile after it is in place, as shown in Fig. 109.

STEEL PILING.

Within the last few years there has been a startling advance made in the construction of piling, due chiefly to the advent of the steel pile. One of the best known steel piles is that made by the Interlocking Steel Sheeting Company of Chicago, and called the Jackson pile. Fig. 110 will sufficiently explain the style and use of this pile. Great depths may be obtained by its use, and by using concrete to fill between the channels, it may be made absolutely water-tight. Of course the cost of this pile is great, a linear foot of 12-inch pile, if made of the lightest

channels and I-beams, will weigh 72 pounds, which would make the cost of steel at two cents per pound, about $1.50 per foot. Steel piling costs about $40 per ton f.o.b. factory. However, they may be withdrawn and driven many times, thus bringing the cost down to a reasonable figure. Almost every form in steel has now been worked into sheet piling and patented, but the author gives one to the public which is not patented and

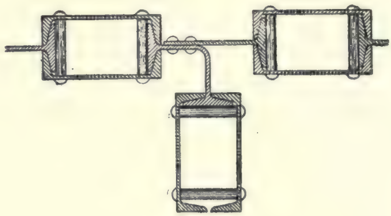


FIG. 110.

which possesses some good features. (See Fig. 111.) In this pile Phœnix columns, boiler plates and angle-irons are used. The columns act as stiffeners to the webs formed by the boiler plates. Any width plate may be used and corners turned by simply bending the plate. The column may be of any size and can be filled with concrete. The weight per foot of a pile 12 inches

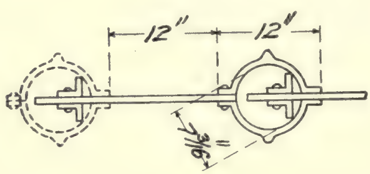


FIG. 111.

wide is 34 pounds, which, at three cents per pound, would cost $1.02 per foot of pile. By using a $5\frac{3}{4}$-inch Phœnix column the pile will weigh 25 pounds per foot.

IRON PILING.

The cast iron pile is used to quite an extent where the iron alone is depended on for supporting the load and resisting corrosion, The smaller iron piles are usually sunk by means of a screw, Fig. 112. The screw has about one complete turn, and

is from 18 to 60 inches in diameter. The shaft usually consists of a piece of heavy shafting. Though in the pile shown in Fig. 113 a hollow 27-inch cast iron shell $1\frac{1}{2}$ inches thick, and made in 7-foot sections was used. These large piles were driven by the

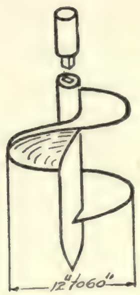


FIG. 112.

Erie railroad for the purpose of sustaining a tunnel under the Hudson River. A large ratchet and pawl (Fig. 113), driven by two hydraulic cylinders, was used for screwing down the

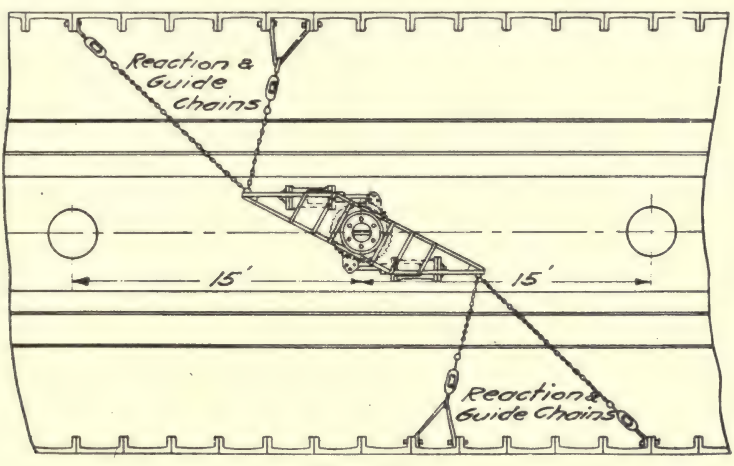


FIG. 113.

pile, each cylinder tested to 1500 pounds pressure per square inch. A dead load of 440,000 pounds, placed on top of the pile was found necessary to cause the 5-inch screw to penetrate 35 feet, and 40 revolutions were made. One pile was driven (ex-

clusive of all such work as placing new sections, etc.) in 10 hours. Under a test load of 500,000 pounds for 15 days and 600,000 pounds for another month, the pile settled a little over ¼ inch.

SAND PILES.

Sand, the great foe to pile driving, is made to act as a pile in some cases where the soil is treacherous. A large wooden or steel pile is driven six or eight feet in the mud, and then pulled out. The hole thus left is filled with sand, well tamped in place. The hole may be dug like a well and then filled. The particles of sand transmit the load equally throughout the area of the hole.

Another form of pile which can be used in sandy and gravelly soils is made by jetting down a 1½-inch gas pipe having perforations at the lower end about ¼ inch in diameter, and then forcing through it a cement grout, the tube being slowly withdrawn. This fills the interstices in the soil and forms a concrete pile of from 12 to 48 inches in diameter. Sand which is too compact cannot be successfully cemented in this way.

BEARING POWER OF PILES.

The bearing power may be approximately determined by the formula (Trautwine):

$$\text{Extreme Load} = \frac{\sqrt[3]{\text{Fall in ft.}} \times \text{Wt. of Hammer in lbs.} \times .023}{\text{Last sinking in inches} + 1.}$$

The safe load would be one-fourth to one-tenth this.

Great caution must be observed in driving piles meant to sustain important loads, which would be injured by settling. A pile may drive into sand with great resistance but under a steady load settle rapidly. The earth's crust is full of strata of varying density and unless test borings are made down past the foot of the pile, it may be resting immediately over a strata of silt or quicksand.

Maj. J. Sanders, United States Engineer, gives the following formula for obtaining the bearing power:

$$P = \frac{W\ n}{8\ d}$$

where P = *Safe* load on pile in pounds.
W = weight of hammer in lbs.
n = fall of hammer in inches.
d = penetration in inches caused by each of the last few blows.

The objection to this last formula is that the selection of the

factor of safety is not left to the engineer. However, as his experiments were made in river mud on the Delaware River, the factor of safety should be about ten in which case this formula gives a much larger safe load than does Trautwine. The *Engineering News* formula is simple and safe and will serve as a check on other estimates. Safe load in tons is

$$\frac{2 \times \text{wt. of hammer in tons} \times \text{fall of hammer in ft.}}{\text{Penetration of pile in ins. for last blow} + 1 \text{ in.}}$$

When the pile is driven to rock or unyielding hard pan it acts as a column and can be figured as such.

A grillage is placed over the heads of piles to distribute the load evenly on each pile as in Fig. 114. Steel beams may also be used.

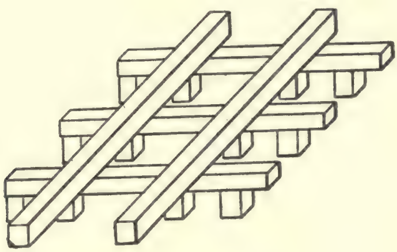


FIG. 114.

A more permanent way to distribute the load is by the use of concrete placed a foot or so deep around and over the heads of the piling as in Fig. 107. Reinforcing should be used so that if any pile or cluster of piles settles, the foundation will hold together.

DRILLING.

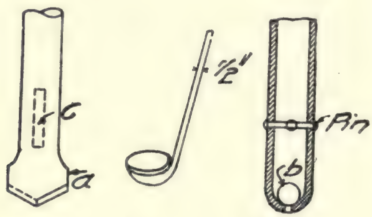
HAND DRILLING.

On small jobs hand drilling is much the cheapest and in fact many times on large work, hand drilling though much slower has been found as cheap as power drilling. The two methods used in hand drilling are *churn drilling* and *jump drilling*. In jump drilling a comparatively light drill is held by one man and struck by one or two strikers. The drill is made of tool steel one inch to one and one-fourth inch in diameter and sharpened as in Fig. 115. Between blows the drill is revolved a quarter turn.

A short drill called a *starter* is used to start the hole. It is given a slightly larger diameter across the bit than the finishing drills, the shoulders at (a) are made parallel and about one-half inch long.

An eight-pound striking hammer is used. A spoon, Fig. 116, or a pump, Fig. 117, is used to keep the hole clean. The pump is made of three-quarter inch gas pipe. A marble *b* is placed in the bottom as shown and by moving the pipe up and down, water and stone dust fill it and it is then emptied by removing from the hole.

When the rock is seamy, the shoulder *a* should be lengthened and for very difficult work a wing *c*, Fig. 115, welded to each side of the drill will help materially. For straight vertical drilling, churn drilling is the most satisfactory. The body of



FIGS. 115, 116, 117.

the drill is made of ordinary iron 1 to 1¼ inch diameter and from six to eight feet long. The drill point or bit is made of tempered steel welded to the bar. In churn drilling the workman (often two) lifts the bar and drops it, giving it a quarter turn each blow. The weight of the bar is relied on to do the cutting. This is the best drill to use for sinking anchor bolts for dams, locks, etc.

The cost of drilling by hand depends on the position of the holes and the character of the rock. The author's experience has been that with jumper drilling, holes 1¼ inch in diameter and 24 inches to 36 inches deep, for anchor bolts in the beds of streams can be drilled in one hour with three men. Trautwine gives seven to eight feet of 1¾ inch hole in granite as a fair day's work for three men and eight to nine feet in marble or limestone.

A churn drill worked by one man will drill about the same amount as a jumper with three men.

MACHINE DRILLING.

Among the machine drills the diamond drill ranks the highest in rate of cutting and depth of hole, but as it is intended more for mineral prospecting or deep well boring, we will not give a

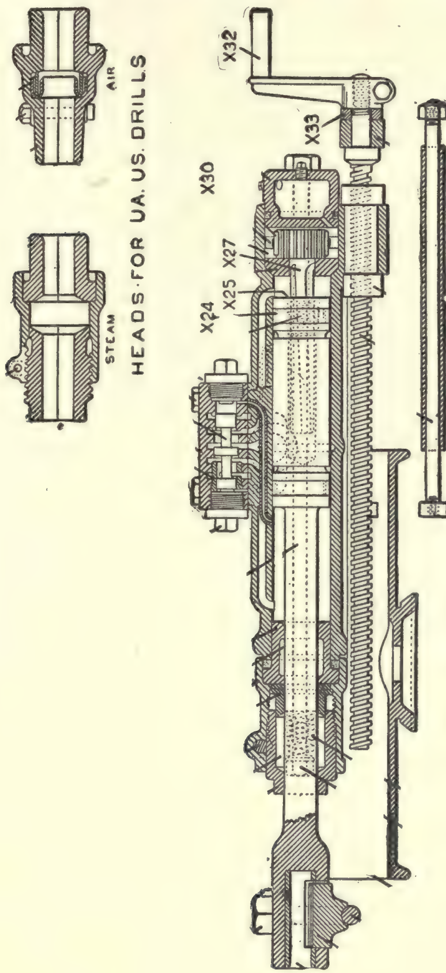


FIG. 118.—Steam or air drill.

detailed description of it. A diamond drill, drilling a $1\frac{1}{2}$-inch hole 200 to 300 feet deep costs about $1250 and will drill one to two feet per hour at a cost of from $1 to $2 per foot.

The drill commonly used is shown in Fig. 118. This drill may be worked with steam or air, the only difference being in the packing of the glands. The price of such a drill varies from \$200 to \$500 depending on depth and diameter of hole it will drill and the depth of feed. The feed is from 12 to 30 inches and this limits the depth of drilling before a longer drill is put in.

If driven by steam the steam pipe will have to be one inch for the smaller drills, $1\frac{1}{4}$ inches for medium and $1\frac{1}{2}$ inches for the large drill having 30 inches feed and drilling a 2-inch to 5-inch hole 27 feet.

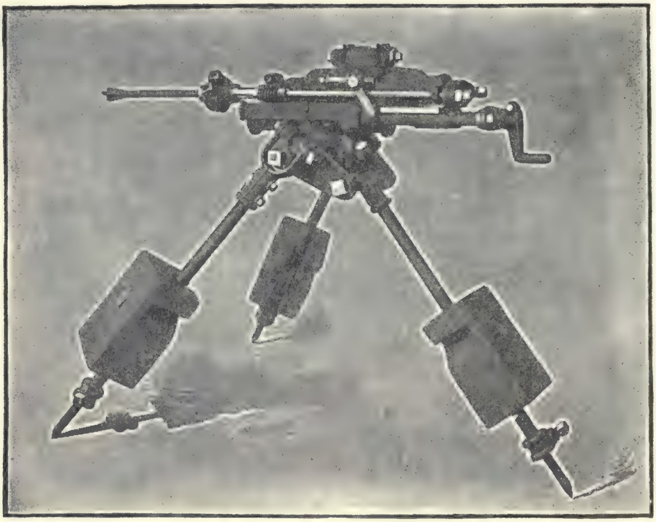


Fig. 119.

If driven by air (see page 192 " Tunnels "), the operation of the drill will be understood by referring to Fig. 118 where X32 is the hand feed, X24 is the piston, X27 the rifle bar which causes the drill to revolve slightly each stroke, X25 the rifle nut which causes the rifle bar to rotate the bar and with it the ratchet wheel X30. Thus each time the piston reciprocates along the bar, the ratchet turns a notch and on the down stroke the piston with the piston rod and drill rotates.

Extra charge is made for the tripod which costs from \$30 to \$80 depending on the size of the weight. Fig. 119 shows

a Sullivan drill all complete except the hose pipe. Hose pipe suitable for the modern drills costs 60 cents per foot. Con-

FIG. 120.

nections will add $4 to this per hose. A mining column, Fig. 121, for tunneling costs about $50.

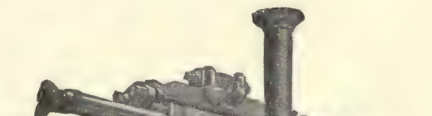


FIG. 121.

For use in hard rock a drill having a bit shaped like a + is best, but for seamy rock where the drill tends to bind it should

be shaped like an X In soft sandstone a chisel bit is recommended. Each drilling machine should have three sets of drills

FIG. 122.—Channeler at work on canal.

of three drills per set. These cost \$3 to \$5 per set for the smaller drills.

FIG. 123.—Channeler at work on tail race.

Each drilling machine requires one man to operate it and takes two to three men to move it. One man can attend the

air compressor plant or steam plant. One blacksmith will sharpen drills for five or six machines if he is provided with special hammers which give the correct form to the bits.

Among the best drills are those made by the Sullivan Machinery Company, Chicago, Ill., the Ingersoll Rock-Drill Company, New York, Burleigh Rock-Drill Company, Fitchburg Mass., and the Graydon & Denton Manufacturing Company of New York.

Only since the Chicago Drainage Canal was built has the channeling machine come into use for canal cutting, but since then several large hydraulic plants have used them. Canals cut with a channeling machine require no lining. Fig. 122 shows a channeler at work. Where it was not necessary to preserve the rock for building purposes, the channeler merely cuts a channel about one inch across and six or seven feet deep along the edge of the canal, and then the rock is blasted out in the usual way. This leaves a smooth surface, unshattered by the explosion. The drill is given a slight slant, as in Fig. 123, so that the general contour of the wall will be perpendicular.

The channeler shown in Fig. 122 costs about $2000 with boiler, but on a large job the canal lining saved will more than pay for it. Such a channeler should cut from 100 to 150 square feet of channels per day, at a cost of twenty cents per square foot.

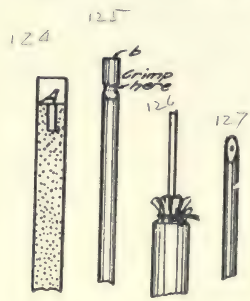
Explosives.

Dynamite is the most common form of explosive, and is used on all kinds of work. It is commonly put up in half-pound sticks wrapped in oiled paper. These sticks are $1\frac{1}{8}$ inches in diameter and 9 inches long. Any strength may be had, the most common being from 30 to 40 per cent. nitro-glycerine, up to 80 per cent.

Dynamite is ordinarily quite safe to handle, though under certain conditions it is extremely unstable, and should be handled intelligently. In small quantities good dynamite may be burned and this widely advertised fact has caused the death of many men. When exposed to the sun or boiling water (as is often done when thawing it out) it rots. This condition usually, but not always, may be detected by the appearance of a greenish tinge. In this state it will explode when burnt or even jarred. In large masses dynamite may be exploded by its own heat while burning.

Dynamite will explode if struck between irons, but not by blows of wood upon wood. A drop of pure nitro-glycerine may ooze from a stick on thawing, and falling three or four inches explode on striking a hot surface. The electric spark and also lightning will explode dynamite. We do not give these defects space here to scare the inexperienced, but to make known that dynamite, when rotting, is dangerous. When in good condition and handled with any degree of care, it is as safe as any explosive. To thaw frozen dynamite, place in a warm place, but do not boil.

To use the dynamite a stick is opened at one end (Fig. 124), and with a punch of the suitable size, a hole is punched two or three inches deep, as at (*A*). Then a cap is placed over the end



Figs. 124 to 127.—Preparing a dynamite charge.

of the fuse, as in Fig. 125, and the end of the cap crimped down on to it. The cap is the most dangerous part of the charge, and great care must be exercised to in no way injure the ends containing the fulminate of mercury. In cutting the fuse off to insert in the cap, use a sharp knife, and be careful not to spill the powder out. Many failures are attributable to lack of powder near the fulminate. The cap and fuse is then pushed solidly into the hole in the dynamite, and the loose paper around the stick is twisted about the fuse and tied solidly with string (Fig. 126). If the charge is under water, or in a damp hole, the exterior around the fuse and string should be coated with axle grease, soap, or some such water-proofing compound. The charge is now placed in the hole and tamped in on top, filling the hole. Dry sand is the best for tamping, but rock dust is also good. The end of the fuse is now slit (see Fig. 127), and the charge is ready for firing.

When a charge fails to explode, great caution should be observed in going near it, as often the fuse hangs fire for two or three minutes. Do not tamp the holes with anything but wood. Rather than attempt to dig out an unexploded stick, drill another hole eight or ten inches from it and fire another charge.

The charges may be varied by cutting the sticks into different lengths. Deep drilling is necessary to produce the best results.

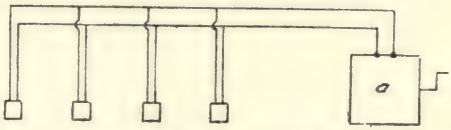


FIG. 128.—Connections for electrically firing dynamite.

The harder the material, the higher the per cent. of nitro-glycerine used. For soft clay, the holes are bored with an ordinary two-inch auger and giant powder used, or mild 15 per cent dynamite.

On large work, or where all the charges must be fired at once, the charges are all connected to an electric circuit as in Fig. 128,

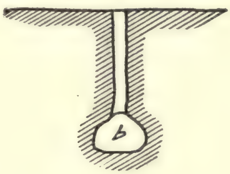


FIG. 129.

by turning the handle of the magneto (*a*) the charges are all exploded. A special cap is used. This method is especially good for under-water work.

When it is desired to get in an unusually heavy charge, a small charge, called a " squib," is first fired, making a cavity, as at (*b*), Fig. 129. This is then filled with dynamite taken from the paper cases and tamped in. For work under a building, and where it is desired to avoid jarring, about two inches of a stick is fired at a time, the stronger grade of dynamite being used.

To blast ice, the charges are placed several feet under water. Large boulders may be cracked by simply placing the charge on top and resting a heavy stone on the charge. Old piling may be cut off under water by boring a hole partly through and sticking in the dynamite. The author once had to drive sheet piling where the sand was filled with saw logs in some places to a depth of eight feet. A place was cut through these by driving two-inch gas pipes, as shown in Fig. 130. The log was first located with a ½-inch rod; in the lower ends of the pipe were plugs. When the pipes were driven the plugs were rammed out and the charges placed in the pipes. A rammer then held the charges in while the pipes were withdrawn. It often required two men one day to cut off a large log, but all were finally cut.

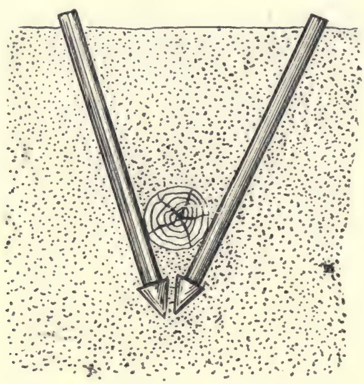


FIG. 130.

Jovite is a more modern production than dynamite, and possesses several advantages over that explosive. It will burn without exploding; jarring will not set it off. It will not rot and become dangerous. It can be hammered iron on iron with perfect safety. For the same strength it is lighter than dynamite. It may be dropped on hot iron. Lightning will not explode it. It contains no liquid, as does dynamite. It *cannot* freeze. A stick may explode within two inches of another without exploding the latter.

Dynamite gives off poisonous fumes, which cause severe headaches. It has affected the author to such an extent that walking was impossible. It has even caused death. Therefore,

when working with it in tunnels or deep cuts, the workmen do not return to the work immediately, but wait for the clearing away of the fumes. Jovite does not give off injurious fumes, and therefore much time is saved in its use.

Jovite is put up in sticks, the same as dynamite, or in bulk. It is graded as: No. 1, which has the strength of 20 per cent. dynamite; No. 2 is equivalent to 40 per cent. dynamite; No. 3XX equivalent to 60 per cent dynamite. It is exploded in exactly the same way as dynamite. In bulk form it may be used to fill into cracks and seams, thus saving much drilling. To get a lifting effect where it is desired to produce quarry stone, No. 1 is used, and where the stone is to be broken up into small pieces, No. 3XX.

For under-water work jovite possesses no especial advantage over dynamite, except in its safety before using. It must not be left in the water any great length of time, as the nitrate of soda leaks out. It can be procured put up in water-tight bags.

The cost of jovite is slightly less than dynamite on account of its lesser weight, the price per pound being about the same.

Ordinary electric fuses with a cap attached and with from four to eight feet of wire for each fuse, cost from $3 to $4 per 100. A blasting machine which will fire 20 charges costs $25. Dynamite costs about 14 cents per pound for the lower percentages of nitro-glycerine, and up to 16 cents for 60 per cent. nitro glycerine.

Cableways.

In building long dams the cableway is the best of all methods for handling materials. There are innumerable systems of cable tramways now in use, but the following are the fundamental types.

Fig. 131 shows a splendid arrangement for making large fills. There are a number of cars *A*, or skips which travel around on the stationary cable *B*, being moved by the traction rope *C*. The terminal *D* is the same at both ends, one or both being used as filling stations. The cars can be loaded automatically as they pass around the terminal and automatically dumped at any point along the cable. Intermediate supports may be placed between the terminals. By adding another rope which is carried around with the car or skip, and a set of falls the car may be stopped at any point and lowered.

Fig. 132 illustrates the most common form used for constructing dams, etc. The carriage is pulled along the cable and may be lowered at any point by a man stationed on the shore. Loads as high as 10 to 20 tons may be handled on spans of over 1000 feet. By making the towers movable the whole field of operation may be covered.

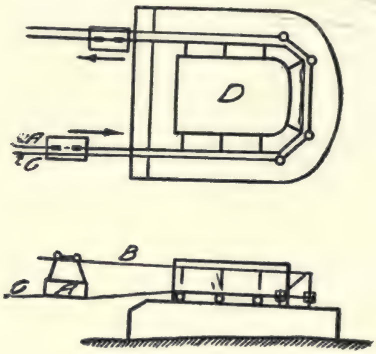


FIG. 131.

FOR SMOKE-STACK GUYS, TROLLEY-LINE SPAN WIRE AND OTHER PURPOSES. COMPOSED OF SEVEN CAL. STEEL WIRES TWISTED TOGETHER.

Price in cents per 100 feet.	Diameter in inches.	Weight per 100 feet in pounds.	Approximate breaking strain in pounds.
315	1/2	52	8,320
250	7/16	40	6,000
200	3/8	30	4,700
160	5/16	22	3,300
115	1/4	13	1,750
80	3/16	8	1,000
60	5/32	5	700
45	1/8	3.50	375
35	3/32	2.25	320

Fig. 133 shows a very good plan where a cheap cableway is desired. By simply varying the elevation of the boom fall blocks, the bucket or skip may be made to run out over the cable and back again by aid of its gravity alone. The dimensions given are for a cableway built by Parker & Flynn of Waterford,

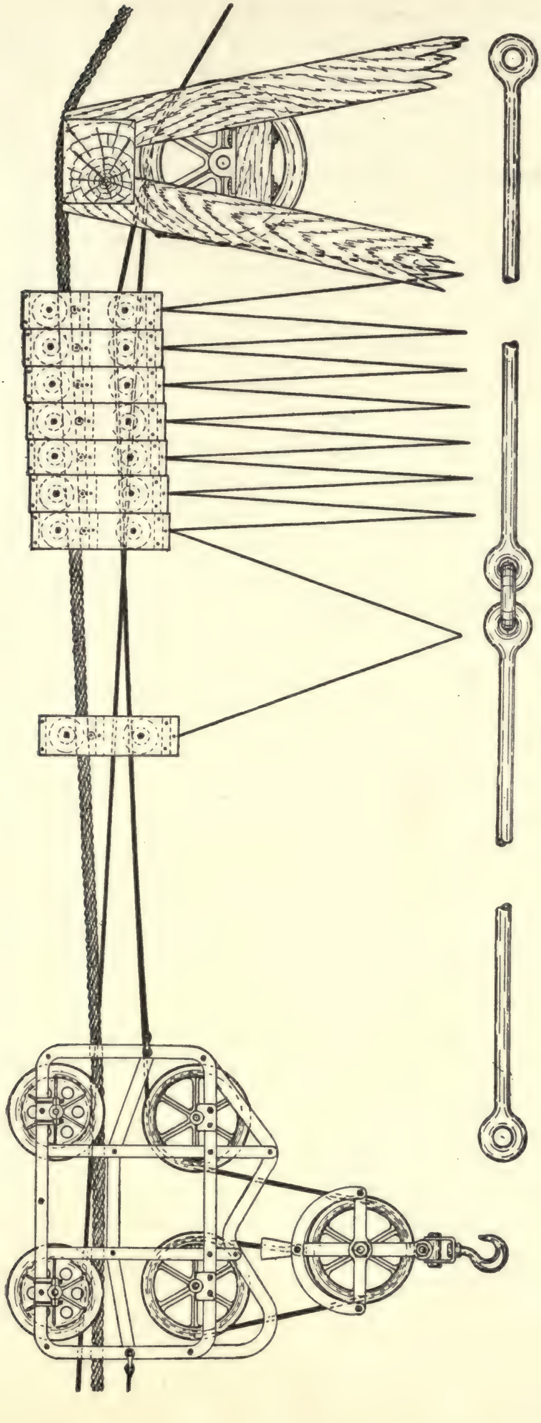


FIG. 132.—Cable way.

N. Y. The total span was 900 feet, and the load five cubic feet of wet concrete. The cable was a $\frac{3}{4}$-inch steel hoisting cable. This plan could be well adapted to dam building. A set of falls would be carried by the skip so as to permit the lifting and lowering of materials. There being no heavy towers, it would be an easy matter to shift the cable up or down stream.

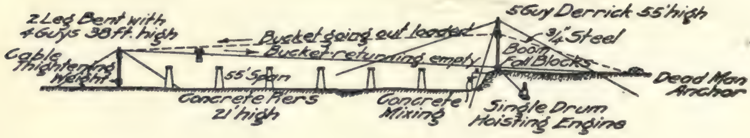


FIG. 133.—Gravity cable way.

The Trenton Iron Company give the following methods of calculating the deflection in cable spans.

The deflection of the cable alone without load at any point x is (See Fig. 134)

$$h = \frac{m \times n \times w}{2\,t} \tag{1}$$

$$\text{for } m = n = \frac{S}{2} \text{ (at the middle)}$$

$$h = \frac{S^2 \times w}{4 \times 2\,t} = \frac{w\,S^2}{8\,t} \tag{2}$$

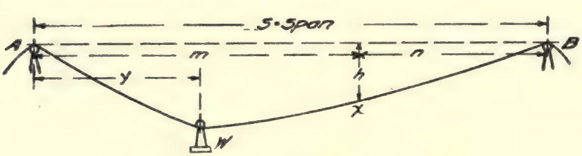


FIG. 134.

wherein h is the deflection in feet; m, the distance in feet from the point x, under consideration to one support; n, the distance in feet to the other support; y, the distance from the load to the nearest point of support; S, the distance between supports in feet w the weight of the cable per foot in pounds; W the load in

pounds and t the tension in the cable in pounds per square inch.

The deflection due entirely to load is

$$h = \frac{W.n.y}{S\,t} \tag{3}$$

for

$$n = \frac{S}{2}$$

$$h = \frac{W\,y}{2\,t} \tag{4}$$

and for

$$y = \frac{S}{2}$$

$$h = \frac{W\,n}{2\,t}$$

(5)

for

$$n = \frac{S}{2}$$

$$h = \frac{W\,S}{4\,t} \tag{6}$$

The total deflection at any point x will be the sum of these two, thus

$$h = \frac{S\,w\,m\,n + 2\,w\,n\,y}{2\,S\,t} \tag{7}$$

for

$$n\,m = \frac{S}{2}$$

$$h = \frac{w\,S^2 + 4\,W\,y}{8\,t} \tag{8}$$

If

$$y = \frac{S}{2}$$

$$h = \frac{w\,m\,n + W\,n}{2\,t} \tag{9}$$

for

$$n = \frac{S}{2}$$

$$h = \frac{w\,S^2 + 2\,W\,S}{8\,t} \tag{10}$$

If the tension is desired, transpose and solve for t, thus in (10),

$$t = \frac{wS^2 + 2WS}{8h} \tag{11}$$

Bridges.

The bridge in some one of its many forms enters so frequently into the design of hydraulic plants that it is deemed advisable to give the design of a few of the more simple forms brief treatment here.

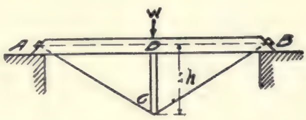


Fig. 135.

One of the simplest trusses is shown in Fig. 135. This truss is adapted to spans of from 30 to 40 feet.

Total compressive stress on

$$A\,B = \frac{A\,D}{D\,C} \times \frac{W}{2}$$

Total tensile stress on $B\,C$ or

$$A\,C = \frac{A\,C}{DC} \times \frac{W}{2}$$

Total compressive stress in $D\,C = W$, wherein W is the total concentrated transient load.

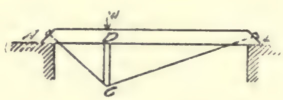


Fig. 136.

These formulas are for a concentrated load: For a uniformly loaded truss W would be divided by 4 in the above.

For a truss with random load, as in Fig. 136:

Total compressive stress on

$$A\ B = \frac{A\,D \times D\,B}{A\,B \times D\,C} \times W$$

Total tensile tress on

$$B\,C = \frac{B\,C \times A\ D}{A\,B \times C\,D} \times W$$

Total tensile stress in

$$A\,C = \frac{A\,C \times D\,B}{A\,B \times D\,C} \times W$$

Compressive stress in $D\,C = W$.

For a truss, as shown in Fig. 137, having equal loads at two points we have:

Total compressive stress on

$$A\,B = \frac{A\,E}{E\,C} \times W$$

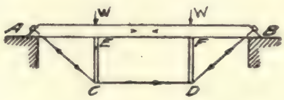


FIG. 137.

Total tensile stress on $A\,C$ or

$$B\,D = \frac{A\,C}{E\,C} \times W$$

Total tensile stress on

$$C\,D = \frac{A\,E}{E\,C} \times W$$

Compressive stress on $E\,C$ or $F\,D = W$

In Fig. 138 is shown the truss Fig. 135, inverted and adapted to a roof truss. As such, the total load of rafters or purlines, supported by the braces $A\,C$ and $B\,C$, produces the same stress

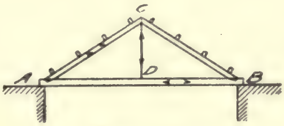


FIG. 138.

as would one-half the load concentrated at the apex C. The horizontal thrust in the rafters at either end equals the tension in the rod $C\,D$.

Any of these trusses may be turned over, in which case the compression members become tension members.

When the spans are longer than about 40 feet a truss, having a number of panels, as in Fig. 139, is used. This is the Burr truss, and as shown here has five panels.

Four-fifths of the total load of truss and transient load is taken as being divided evenly between the four points of support, C, D, E and F. One-fifth is supported by the abutments.

$$\text{Total compressive stress on } A\,G \text{ or } B\,J = 2\,W \times \frac{A\,G}{G\,C}$$

$$\text{Total tensile stress on } G\,C \text{ or } J\,F = 2\,W.$$

$$\text{Total compressive stress on } H\,C \text{ or } I\,F = W \times \frac{H\,C}{H\,D}$$

$$W = \frac{\text{Total wt. of load and truss}}{\text{Number of panels}}$$

The diagonals $H\,E$ and $I\,D$ receive no stress unless the truss is unequally loaded. The rods $H\,D$ and $I\,E$ each sustain a

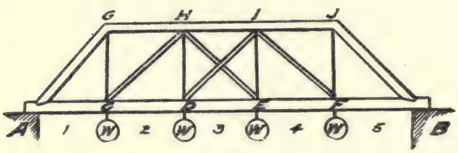


FIG. 139.

$$\text{Total compressive stress} = W.$$

$$\text{Total compressive stress in } G\,H \text{ or } I\,J = 2\,W \times \frac{A\,C}{G\,C}$$

$$\text{Total compressive stress in } H\,I = 3\,W \times \frac{A\,C}{G\,C}$$

$$\text{Total tensile stress in } A\,C \text{ or } F\,E = 2\,W \times \frac{A\,C}{G\,C}$$

$$\text{Total tensional stress in } C\,D,\ D\,E \text{ or } E\,F = 3\,W \times \frac{A\,C}{G\,C}$$

For large trusses the cords and even the braces frequently have to be built up. A properly built up timber is better than a solid timber of the same area, for though the strength may be less, it will last longer, the interior being ventilated.

Fig. 140 shows some of the splices used by the Pullman Car Mfg. Co. and recommended by the Master Car Builders' Asso-

ciation. Fig. 141 shows a method of building up a chord composed of a number of planks, *a*, *b*, *c*, *d*, *e*, etc. The leaves, *x*, are made of hard wood one third the thickness

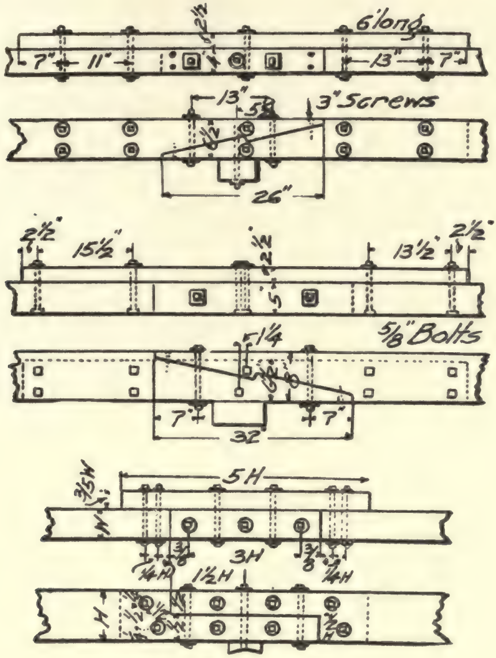


FIG. 140.

of the plank and of a width three times its own thickness. The cast iron plates, *O*, are placed as shown, being staggered so as to weaken the chord as little as possible. The strength of such

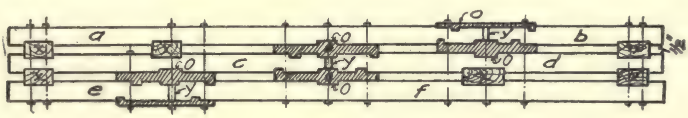
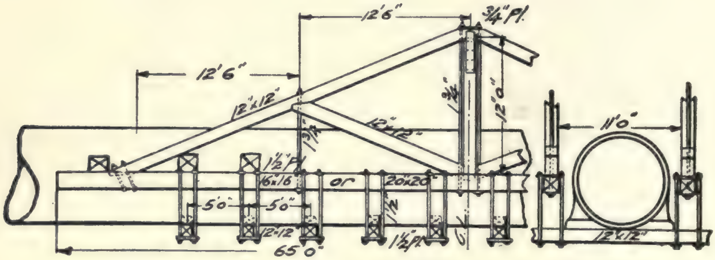


FIG. 141.

a beam should be taken as about two thirds that of a solid beam of the same net area. Where the ends of the plank come together a wedge, *y*, is driven.

In case of an emergency where quick work is necessary, the plank may be spiked together. Wedges are driven between the ends. In all cases two bolts should be used, each having a factor of safety of about three. Then if one rod should have a flaw the other will hold the truss. All trusses should be slightly bowed up at the middle of the span.

Fig. 142 shows the construction of a bridge for carrying a penstock.



Fig, 142.

Suspension bridges are often used to carry penstocks or heavy transmission lines across valleys. Referring to Fig. 143, $A\ B$ is called the chord; $A\ G$ and $B\ F$ the backstays, and $C\ E$ the deflection. For all practical purposes the curve $A\ E\ B$ may be called a parabola in which case,

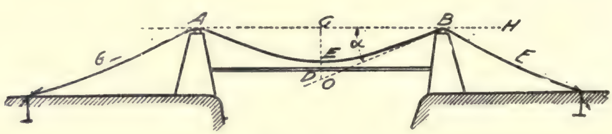


Fig. 143.

$$\sin \alpha = \frac{2\,C\,E}{\sqrt{(2\,C\,E)^2 + (A\ B/2)^2}}$$

Or if one wishes to get this angle from the drawing, lay off $O\ E =$ to $C\ E$ and connect $O\ B$. Then with the protractor get the angle $A\ B\ O$.

Stress on all the suspension cables at A or B equals one-half the entire suspended weight of clear span and its load divided

by sin α. The stress on the backstays will be equal to that on the main cables if angle $C B O = H B F$, and the stress on the pier will be vertical and equal to the entire suspended weight of the clear span and its load. For this reason every effort should be made to so locate the anchorage of the backstays that the angle $C B O = H B F$.

In case these angles are not equal, as in Fig. 144, to get the stress and its direction on the tower lay off to scale on $B F$ and $O B$ the stress on the main cables as $B m$ and $B n$. Then complete the parallelogram of forces $B m P n$. Then $B p$ will represent the magnitude and direction of the stress on the tower. The deflection may be from one-tenth to one-fifteenth of the span.

One of the greatest dangers to suspension bridges is the effect of the wind, causing undulations. In the case of a

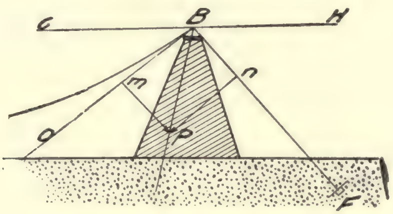


FIG. 144.

bridge carrying a penstock this would result in causing leaks. To guard against this cables are used to guy the span in both the horizontal and vertical planes.

COFFER DAMS.

More money has been needlessly wasted on coffer dams than upon almost any other one structural detail. Coffer damming comes entirely under the cost of building, and the contractor is tempted to either build too cheaply or too expensively, depending a good deal on the size of the bond.

Coffer dams may be divided into classes:

(1) Sand bag; (2) Horse; (3) Bridge; (4) Pile.

Case (1). In many instances the sand bag seems to be the only suitable means and it is always a costly one. Great care must be exercised in selecting the sacks. Feed, bone, meal,

etc., sacks unless perfectly clean, will soon decay and cause trouble. It pays to get good sacks. An 8-ounce, 48-inch burlap sack is a good size. The sacks should be filled only about three-fourths full, so that they may be well packed into the recesses. It takes about 75 48-inch burlap sacks to make a row (when laid side by side), 100 feet long. When tied, such a sack is about 30 inches long.

In building a coffer dam to hold a head of water, the dimen-

Fig. 145.

sions must be figured out to resist that head. The base must be broad and the sacks laid with all possible care. Of course, it often occurs that the sacks must be dumped in without regard to packing or breaking joints, in which case a broader base is required. A good rule to go by is to use two sacks end to end for the top row and widen out half the length of a sack on each side, with each tier of sacks. This will give suffi-

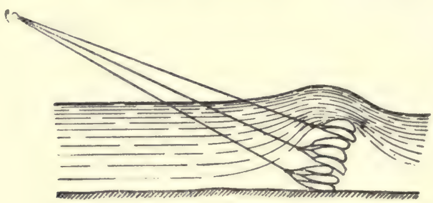


Fig. 146.

cient base for all ordinary purposes. On sand bottoms it is usually necessary to pave the bottom with sacks as in Fig. 145, so that the water falling over the dam before completion will not undermine the coffer.

When the dam is to be water-tight, a fill of earth or sand will have to be made on the up stream side.

It is sometimes quite difficult to make a sand bag stick, on account of the current, especially when closing up the dam.

The author was once called upon to build a coffer dam on a sand bottom where the current had a velocity of about 500 feet per minute. At first six or seven sand bags were tied together and thrown in, but the current immediately swept them out, Finally the method shown in Fig. 146 was adopted. A heavy rope was let down to the dam with seven sacks tied to it. The current kept these suspended in mid-water. Then another rope was let down with seven sacks tied to it. These weighted the first seven down to bottom fairly well, but a third rope and cluster of sacks was necessary to hold them securely. Now the rope to which the first seven sacks were tied was cut loose and used for seven more sacks. In this way the dam was successfully built.

Sand bags should always be sewed as in Fig. 147, thus forming

FIG. 147.

"ears" by which they may be handled. It is also a better shape for use in the dam.

For estimating the number of sacks necessary to build a coffer dam, assume that each sack occupies $1\frac{1}{2}$ cubic feet of space and then add at least ten per cent. for lost sacks.

The sacks should not be filled until ready to place under water as they quickly decay if left in the air and moisture.

According to the author's experience, a coffer dam entirely of sand bags is the most expensive and requires the longest time to build.

Case (2). The horse coffer dam is the invention of E. R. Beardsley, and in the author's experience with hundreds of coffer dams, it has always proved to be the cheapest, surest and most quickly built. The method of construction is as follows:

A strong horse *a* (Fig. 148) is built of logs or squared timber. (For small dams only two legs are required, but for heavy work four will be necessary, as shown in Fig. 148.) If the bottom is of sand, a plank, *b*, is fastened under the legs as shown, but where the bottom is hard, simply the braces, *e*, which are made of boards, one inch by six inches, are spiked along the sides. The legs are mortised into the log and stand at right angles to it.

Across the horses are placed stringers, *c*, and on these are laid the poles, *d*. In placing the poles care must be exercised (especially with sand bottoms) to avoid concentrating the current at any one point. Begin at the ends and work toward the middle, placing the poles a slight distance apart, and afterwards filling up the spaces left. Place the *small* end of the

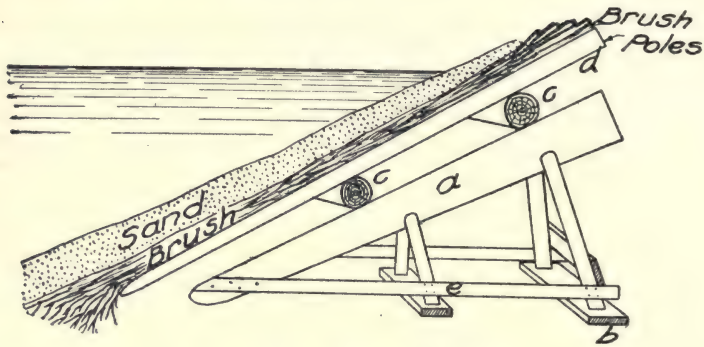


FIG. 148.—Horse coffer dam.

pole up stream. Brush are next placed on the poles, the *green bushy* ends being placed well up stream. When the brush is thickly and evenly distributed entirely across the dam, the ends of the dam are made safe by using plenty of hay, cane, corn stalks, etc., and earth.

The bottom or up stream edge of the dam is next made secure. If the ends and bottom are not made safe, all the work will be lost, and having made them safe it is then an easy matter to complete the job. On sand bottoms it is necessary to pave the bottom with sand bags before the coffer dam is commenced. The horses are usually placed about ten feet apart. Such a dam is comparatively safe from floods and can

pass water over the crest. In fact, it is almost impossible to wash out such a dam. The author built a dam like this on the Elkhorn River, Nebraska, where the bottom was of the worst possible sand, the current had a velocity of over 400 feet per minute, and the water nine feet deep. The dam was 60 feet long and cost as follows:

Poles	$ 8.00
Cutting Poles	7.00
Cutting Brush	6.00
Six Horses	9.00
Filling Sand Bags	150.00
2000 Burlap Sacks	100.00
	$280.00*

Where the bottom will permit, plank may be used part way

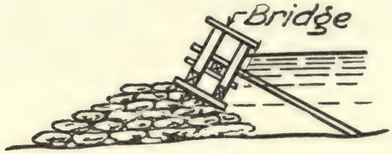


FIG. 149.

in place of the poles and brush. It often happens that planks can be used to build part way out from the ends and then poles and brush to close up with.

Case (3). It sometimes occurs that the current is so swift and the water so deep that the horse dam is impracticable. In such cases a truss bridge may often be used. Figs. 149 and 150 illustrate this method. Piers must first be sunk at proper intervals, say 40 feet, and a truss bridge built upon them. With the truss as a foundation the dam is built the same as the horse dam. In a coffer dam built as shown in Fig. 149, the water was 16 feet deep and the sand bottom was all filled with bark from saw logs which had accumulated for years.

Fig. 150 shows a three truss bridge floating on the water above a dam in which there was a break 40 feet long and eight feet deep. This truss was floated across the break and successfully closed it, brush and gravel being used to stop all leakage.

*This is $4,66 per foot and should be the maximum cost. Small dams on gravel bottom cost from 50c to $2,00 per foot.

Case (4). The coffer dam commonly used by engineers is shown in Fig. 151. Sheet piling *a* is driven in two rows the distance between the rows being about three-fourths the depth of the water. The range piles *b* are driven first and the string-

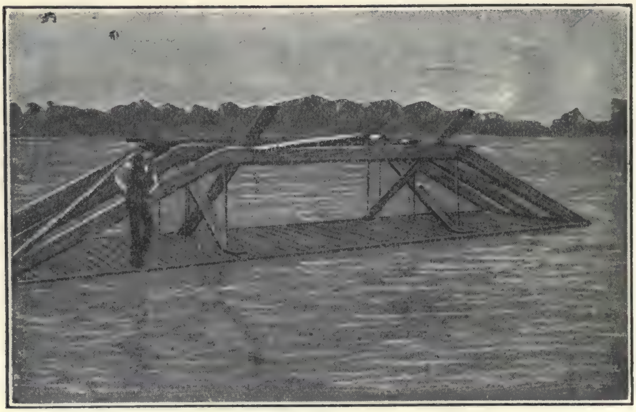


FIG. 150.—Bridge built to stop break in dam.

ers, *e*, placed for guides to the sheet piling. Sand makes the best filling as any leak immediately makes its location known. Clay puddle is often used. The bottom of the coffer should

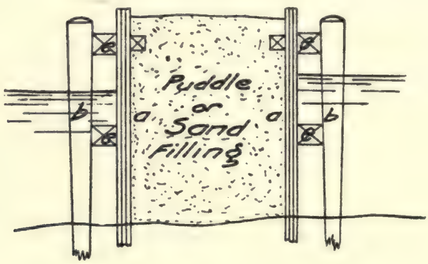


FIG. 151.

be free from all loose stone and gravel, else there is apt to be leakage at that point.

This type of coffer dam, when properly built, will cost from $5.00 to $10.00 per linear foot. It is a difficult dam to build in swift water and is not suited to sand bottoms. However, for certain conditions and places it is a good type.

For rock bottoms the plan shown in Fig. 152 may be used either for a caisson or for a coffer dam. The stringers, *a*, are bolted together with the plank, *b*, placed at each bolt. These planks are given a taper and the widest end is placed down. Slots are made where the bolts come so that the plank can be driven. The skeleton may then be floated into position and the plank, *d*, driven. In this way a tight fit may be made with the bottom. The fill is now made of sand or clay puddle.

The planks *c* are bolted to the frame to hold the skeleton in position while being floated in place. They also hold it off

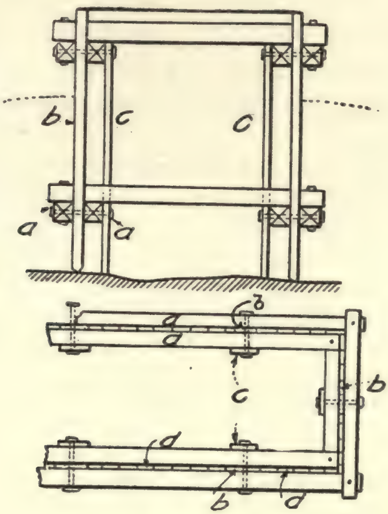


Fig. 152.

of the bottom while the plank are being driven. When used as a caisson the fill is made around the outside as in dotted line.

Caissons.

Where it is necessary to excavate below water a caisson is used to keep out the water. The building of wheel pits almost invariably makes this necessary. For small areas and shallow depths a common earthen coffer dam as shown in Fig. 153 is sufficient, but for depths of three feet or more and when the width is not more than 18 or 20 feet it will pay to build a timber caisson as in Fig. 154.

The sheeting should be edged and given a slant as shown in Fig. 154 so as to utilize the pressure of the water and soil. If the sides were made parallel the pressure at the bottom would tend to spring the caisson as shown in Fig. 155, and in this form the water and loose earth on the outside would tend to force it out. If, on the other hand, the caisson has the form

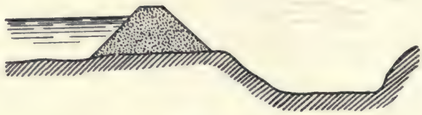


Fig. 153.

shown in Fig. 156, the water and earth act as shown to hold it in place and much less trouble will be experienced in keeping it tight.

For excavations of large area the author has designed a caisson which possesses some good features. Fig. 157 shows an end view and plan. At 12 foot intervals bridge trusses

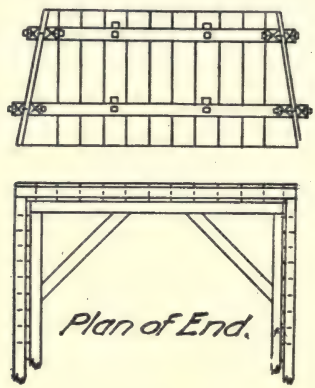


Fig. 154.

are placed across the caisson. The only limit to the width is the limit to the length of a bridge truss. Braced against the top chord of the truss is a 6x6 or 8x8 strutt, *A*, which rests on the stringer, *D*; on the under side of this strutt is spiked or bolted a cleat which receives the pressure of the stringer. At *F* cleats are also bolted to the lower chord. The stringers

D and G are bolted to every fourth or sixth plank, slots being made in the plank to allow driving. At *C* a platform is laid loosely for a wheeling platform (when wheelbarrows are used), and there should be sufficient head room between the truss chords to permit a workman to walk along with a load.

In most soils large weights of rock or timber will have to be piled on top of the trusses to cause the caisson to settle, and in this case the trusses will have to be designed to sustain quite a load. The pump is mounted on one corner of the caisson.

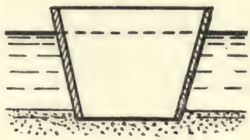


FIG. 155.

All the timber can afterwards be used for matting or concrete forms.

In the case of a caisson of this design, 80 feet long, 40 feet wide, and 8 feet deep, it took 12 men eight days to construct and install it. This does not include excavations.

With this caisson any depth may be attained through any soil. Nothing is in the way of the workmen and nothing has

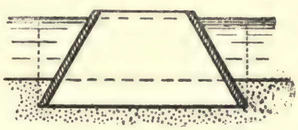


FIG. 156.

to be moved from the time the sinking commences until it is completed. A mat may be laid all over the bottom without moving a pile or a timber.

Sometimes the concrete walls are laid directly against the sheeting, it forming one side of the form, so as to avoid leaving cavities or loose earth around the outside of the wall.

A method usually adopted by engineers is shown in Fig. 158. This is an end view and shows the soil before excavation is started. Round piling is driven at equal intervals lengthwise

and crosswise and about 14 feet apart. The object of these piles is to hold up the braces *a* which resist the inward pressure of the water. Tight sheet piling *b* is driven so as to enclose the excavation. The workmen excavate the soil around among the piles, and as the level is lowered more braces are placed as at *c*. When the desired depth *D E* is reached the bottom

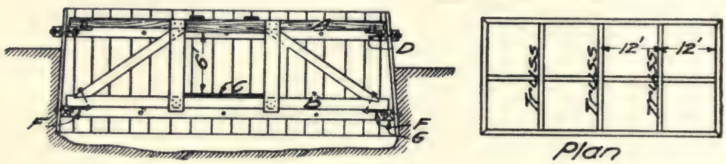


FIG. 157.

is concreted around the piling and the walls built up. Then the piles are cut off level with the floor. This leaves a portion of the round pile supporting the floor and usually many other piles are driven before concreting so that the foundation is supported on piling and the weight taken off of the earth under-

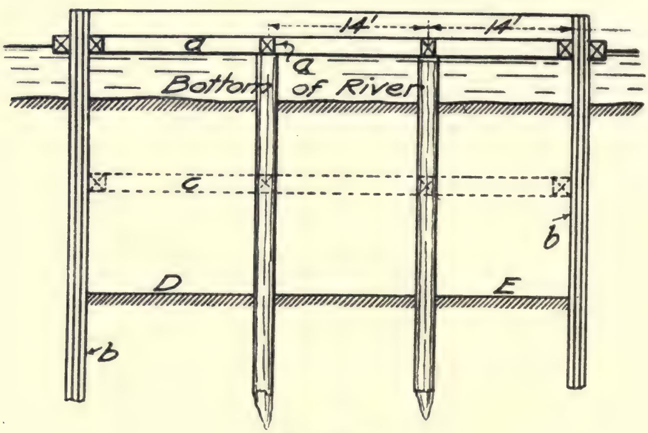


FIG. 158.

neath. This is a very bad thing to do as the earth is sure to settle and a leak will spring between the conrete and the earth. The great Soo plant was threatened with destruction because it was thus built. Water got under the power house and washed out a hole over 100 feet wide and 20 feet deep.

The above caisson using bridge trusses cost about $150 to build and install, not counting materials as they were afterwards used. The same caisson if built as in Fig. 158 would cost at least $800 including the materials which could not be used again. Sheet piling is very difficult to drive so that it will be water-tight and for caisson work it is very essential that it should be so.

Usually the caisson will have to be protected from the river current by a coffer dam.

Costs.

The author has found that blue clay excavation inside caissons and where the clay is too hard to shovel will cost about $1.50 per cubic yard, when wheeled up an incline to an elevation of from 15 feet to 25 feet. This is the average cost of all the clay from water's surface to a depth of nine to 12 feet below, labor at $1.75 per day. It does not include cost of caisson. When taken from the caisson in skipps the cost is half that given above.

Sand, clay and gravel, mixed, taken from a caisson 7½ feet deep and not deposited higher than the top of the caisson costs, $1.25 per cubic yard to excavate by pick and shovel and take out by barrows.

Gravel and quicksand taken from a caisson five feet deep cost 65 cents per cubic yard.

The total cost of sinking a caisson 36 feet wide and 80 feet long to a depth of nine feet in hard clay and laying a mat consisting of two layers of plank on sills, over the entire area, was $1556. Another caisson 24 feet wide, 34 feet long, and 7½ feet deep cost $250 to sink in gravel and clay. Another 24 feet by 18 feet by six feet cost $200 to sink in gravel and silt. Another 100 feet by 40 feet by 14 feet sunk in gravel, sand and clay cost $2200, a skipp being used to elevate the materials to the top of a 25-foot bank. The two largest caissons were built as in Fig. 157 and the others as in Fig. 154. The cost of operating the pump and all labor is included in the above costs, as well as the laying of the mat.

Pumps.

The centrifugal pump is one of the most useful pieces of machinery which is used in hydraulic construction work. It can

be used for many different purposes; is reliable, is simple in construction and will stand a lot of rough usage.

It is often employed in dredging work and will remove all sorts of soil, stone, etc., without being injured to any great degree. Some pumps are lined with steel to better resist the action of sand and grit. The Morris Machine Works recently patented a pump which has a removable and adjustable lining. The pump is made in sections and is particularly suited to mountain work where mule transportation is necessary.

Up to a few years ago centrifugal pumps were recommended for low heads only, but the new designs of multiple stage pumps driven at a high speed by electric motors or steam turbines can compete with any high head reciprocating pump on the market to-day.

A centrifugal pump is a peculiar piece of apparatus and does not behave like an electric generator as many engineers are wont to assume. In ordering a generator it is always good policy to get a little larger machine than is needed for the average load, but this is not so with the centrifugal pump. Always give the builder the *exact condition* under which the pump is to work.

It very often happens that the purchaser orders a pump of larger capacity or higher head than he needs so as to be on the safe side as he puts it, and is surprised to find that his motor or engine takes an astonishing amount of power and perhaps won't drive it at all. The formula for the power necessray to drive the pump is

$$W = \frac{H\,Q\,62.5}{\eta\,33000}, \text{ in horse power,}$$

wherein H is the head in feet; Q the volume discharged in cubic feet per minute and η the efficiency.

The efficiency of the pump is best when the water passes through it at the velocity calculated by the builder, but for any other velocity above or below this the efficiency will fall off roughly as shown in Fig. 159.

If the static head is constant and it is desired to vary Q, the effective head must be varied either by varying the speed or by throttling the exhaust. Throttling causes a waste in head and reduces the efficiency, while reduction in speed reduces

the efficiency of the steam engine and is impossible with the constant speed *a c* motor. Referring to the curve it is seen that horse-power is practically constant for loads below normal, so that the efficiency is very poor at low loads. The horse-power W, under these conditions, is expressed by

$$k \frac{Q}{\eta} = W, \ k = \frac{H.62.5}{33000}$$

From this equation it is seen that Q and η decrease together, therefore the reduction in power is slight.

On the other hand suppose that a pump designed for a 20-foot pressure head is used to dredge under a 2-foot pressure head, and that it is directly connected to a high-speed steam engine or an electric motor. The reduction in head will cause an

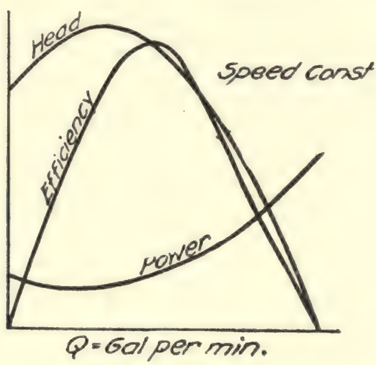


FIG. 159.

enormous increase in Q, which will clog the pump and if driven by an engine bring the engine nearly to standstill; if driven by a motor will blow the fuse on a continuous current motor, and cause an induction motor to run over the breakdown point and stop. For all-round work a variable speed motor, which is provided with a compensating winding or inter poles, should be used. These motors can be efficiently varied over a large speed range and at the same time give a constant output.

Another disadvantage in having too large a pump is that it cannot be run continuously; for instance, in pumping out a caisson it will have to be continually started and stopped, and may cause trouble due to the priming. A globe valve placed in the discharge does away with all priming trouble.

In important work there should always be two pumps, so as to have one in reserve. Tables of speeds and powers for different heads are obtainable from any of the makers.

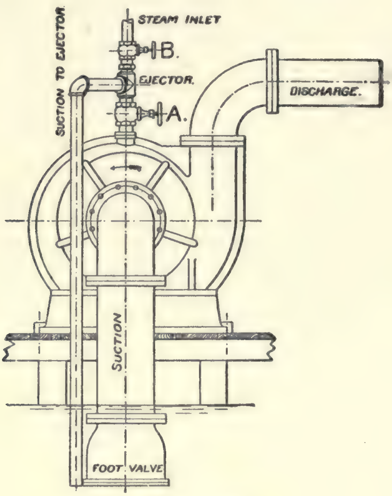


FIG. 160.

A velocity of 10 feet per second should not be exceeded in the suction or delivery pipe. The suction pipe should be perfectly

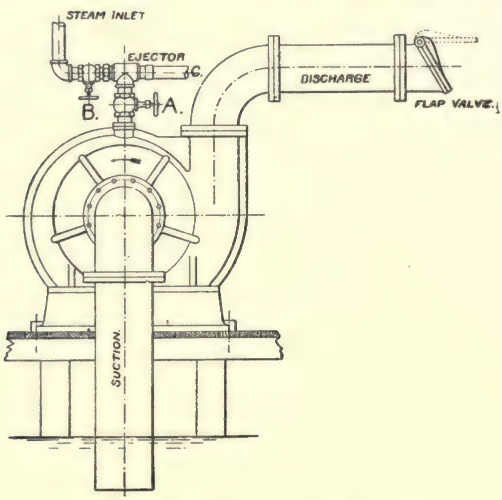


FIG. 161.—Centrifugal pump.

air-tight. A gate valve should be located near the pump in the delivery pipe. A coarse screen should encase the lower end

of the suction pipe to prevent stones entering and getting under the valves.

Before starting, the pump must be filled with water. In order to make this possible the pump must be supplied with some kind of valves in the suction or delivery pipes. If the valve is in the discharge the air must be exhausted and the water from the suction allowed to fill the pump. The air can be exhausted with a hand air pump or with an ejector operated by steam, compressed air or water under high pressure. On the other hand if the valve is in the suction the pump must be filled with water from above. This may be done with pails, with an ejector or by allowing water to run in from a barrel or tank. Figs. 160 and 161 show the connections for an ejector used respectively with a valve in the discharge and suction line.

Pumps supplied with flap or foot-valves will hold their prime for quite a while when not in use, at least long enough to premit the use of a pump in intermittent work, such as filling a tank controlled by a float. The delivery valve should not be opened until the pump is up to speed.

Hydraulic Ram.

Another simple piece of apparatus which is often used to raise water is the hydraulic ram.

The simple and effective operation of this machine, and its great durability, render it a most useful and valuable apparatus for elevating water, and conveying it to almost any desired distance.

It is practicable where the spring or brook is only 18 inches higher than the ram; yet as the height increases, the more powerfully the ram operates, and its ability to force water to a greater elevation and distance is correspondingly strengthened. The relative height of the spring or source of supply above the ram, and the elevation to which it is required to raise, determine the relative proportion between the water raised and wasted—the quantity raised varying according to the height it is conveyed with a given fall; also the distance the water has to be conducted, and the consequent length of pipes, have some influence on the quantity delivered at the point of discharge, as the more extended the pipes through which the water has to be forced by the ram, the more friction there is to overcome by additional

effort on the part of the machine; notwithstanding rams are frequently and successfully employed for driving water a distance of 100 to 200 rods, to an altitude of 100 to 200 feet above the ram, and severer trials than this even, testify to the indispensibilty of this almost automatic device. A fall of 10 feet from the brook or spring to the ram is abundantly sufficient to raise water to any point less than 150 feet above the location of the machine, while the same amount of fall will also raise water considerably higher, though the quantity of water will be proportionately diminished as the height and distance increase. When the requisite quantity of water is forthcoming from the ram, operating under a certain fall, it is not judicious to give it more fall, for by so doing the strain on the machine is measurably augmented, those parts doing the labor are overtaxed, and the durability of the apparatus impaired and lessened.

For ordinary purposes it is sufficient to say, that in conveying water, say 50 to 60 rods, it may be safely calculated that from one-tenth to one-fourteenth of the water can be raised and discharged at an elevation ten times as high as the fall or one-seventh part of the water can be raised and discharged, say five times as high as the fall applied, and so in like proportion as the fall or height is varied. Thus with a fall of five feet of every seven gallons drawn from the fountain, one may be raised 25 feet or, half a gallon, 50 feet. Or, with 10 feet fall, one gallon of every 14 may be raised to a height of 100 feet, and so in like proportion as the fall or height is varied. (See general rule.)

Where the water is to be forced to any great distance, say more than 75 rods, it is preferable to use a discharge pipe of larger caliber than named in the table.

Several rams can be set so as to play into one discharge pipe—each ram having a separate drive pipe applied from the spring to the ram.

The size of the pipe may be varied in proportion to the distance the water is to be conveyed, as the greater the distance the larger the pipe in proportion to the size of the machine. This, however, applies only to the discharge pipe.

Turns in either drive or discharge pipe should be avoided if possible. When it is impossible to set the ram without having elbows in the pipes, make the elbows as large as may be so as to place as little obstruction to the free and easy flow of the water as is practicable.

These machines are made of iron and brass. The valve stems are made of bronze, which has more durable and lasting qualities than any other composition. The annexed table exhibits the capacity, size, price, etc.

GENERAL RULE.

Multiply the number of gallons furnished by the spring, per minute, by 936; multiply this product by the height of the spring (in feet) above ram; then divide by the height (in feet) between ram and point of delivery. The result will be the number of gallons delivered per day of 24 hours.

The following table gives the capacity of the several sizes of Rumsey & Company rams and the dimensions of the pipes to be used in connection with same.

Table XXXV.

Size of Ram.	Minimum Quantity of Water Required to Operate Ram. Gals. per Min.	Length of Drive Pipe. Feet.	Calibre of Pipes.		Price.
			Drive. Inches.	Dis-charge. Inches.	
No. 2	2	Five to six times	$\frac{3}{4}$	$\frac{3}{8}$	$ 9.00
No. 3	4	height of the sup-	1	$\frac{3}{8}$	11.00
No. 4	8	ply.	$1\frac{1}{2}$	$\frac{1}{2}$	14.00
No. 5	14		2	1	22.00
No. 6	25		$2\frac{1}{2}$	$1\frac{1}{4}$	40.00
No. 7	60		4	2	75.00
No. 8	120		6	$2\frac{1}{2}$	125.00

EMBANKMENTS.

Embankments frequently play an important part in hydraulic work. It stands to reason that the embankment should be as permanent as any other part of the work, yet they are too often scrimped and built by guess.

Too much care can not be taken in thoroughly cleaning the ground where the embankment is to stand, of all grass and loose stone, etc. It must then be well plowed. To neglect this will insure a seepy and dangerous embankment. Avoid making a fill of earth having a large proportion of stone. The earth washes out, leaving a leaky embankment.

Clay or sand are the best materials for this work. Sand has the great advantage of being rat proof. The musk rat is a constant menace to all earth embankments. They start digging two or three feet under water, taking advantage of every root or stone to support the roofs of their burrow, and when into the embankment a few feet, dig upward to get above the water for their nest. They then continue digging downward till they emerge on the down stream edge of the embankment and just under water.

Their work remains concealed until the floods come and then there is a call for quick work. A purely sand embankment, while demanding flatter slopes, is safe against rats because the sand falls in as fast as they dig and thus stops them. Again any seepage makes its location known at once while in a clayey soil a large cavity may be washed out without any warning.

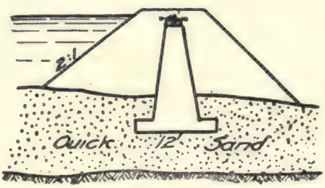


FIG. 162.

A puddle wall in the center (Fig. 162) is often used to prevent seepage, but this is a bad design as it renders useless all that part of the embankment on the up-stream side of the puddle. All the expense in making the embankment water tight should be spent on the up-stream side. Fig. 163 shows the proper section. The clay, when used, is put on after the fill is all in. It should be *thoroughly* damped and *tamped.*

Where the materials can be selected, the most impervious are put on the up-stream side and the coarser and more sandy on the down stream side of the embankment.

At the top the width should be at least 12 feet, which allows for a wagon road and gives about the right proportions. The slope of the sides should be determined as described in page 188. A slope of two to one for the down-stream side and three to one for the up-stream is a common section.

The depredations of the rat may be guarded against by properly rip-rapping. It will not serve the purpose to merely throw in a large quantity of rock, as to do so makes the best possible refuge for the pest. A mink or musk rat will find a dozen fine passages through such riprap, and the mass of loose stone serves to hide the burrow.

A splendid riprap, and one which is not at all expensive, is to pave the up-stream slope with a 4-inch layer of concrete in which wire netting is imbedded. If rocks are used they should be laid with great care so that while they leave no holes that one could get one's hand through, they at the same time do not afford a hiding place.

In a well designed embankment the line of seepage should strike within the base as in Fig. 163. There is no way of predetermining the line of seepage except by building an experimental embankment.

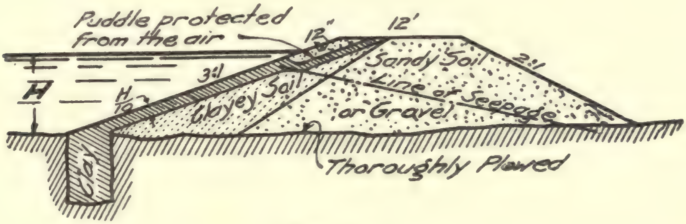


FIG. 163.

CANALS.

In selecting the location for the canal, it must be borne in mind that the longest way round may be the best. The cheapest canal is one dug along a hillside and in such a way that the excavation just forms one bank as in Fig. 164. This possesses certain disadvantages, one of which is the danger of seepage along the line *A B*. However, if the soil is well ploughed before the fill is to be made and the fill thoroughly packed by horses or otherwise (see embankments) there should be no trouble from this source.

A canal of this sort is exposed to the depredations of muskrats, and should be regularly inspected. If there is difficulty in getting the proper slope at *D E* that side may be riprapped or sheeted. Where the soil is treacherous as is the case with

certain clayey loams, a puddled wall (see page 186) should be built. One of the most important items in the design of an earth canal is the selection of the proper angle for the banks. The angle at which the earth will stand without flowing down is called the *angle of repose*, and this angle must be determined

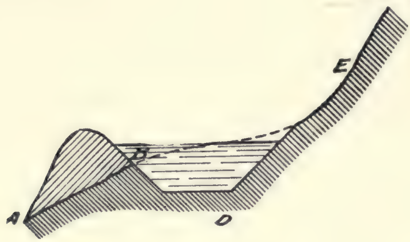


FIG. 164.

before work is commenced. The only safe plan is to take a quantity of the soil from the line of the canal and bank it up in water. The angle which the soil assumes under these circumstances may safely be taken.

Once the author built a long embankment out of what appeared to be good soil for the purpose. In dry embankment

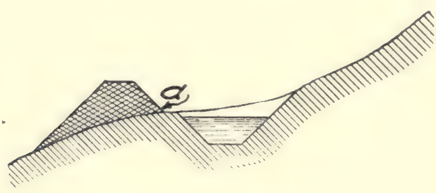


FIG. 165.

it packed splendidly, there being just enough clay mixed with the light loam to make it stand up well But when the pond was filled the embankment ran like oil and it required the most desperate all-night work to keep the top above water. A few practical lessons like this teach the importance of careful pre-

liminary examination. No safe rule can be given for the angle of repose and no two engineers will agree on what it is.

The section shown in Fig. 165 is better than Fig. 164 as the loose earth excavated is above the water line as is the line of greatest seepage. It is a good plan to place the edge *a* of the fill a few feet away from the edge of the canal. This space is called the berm and serves to catch the sluffings from the fill.

In spite of the fact that a lining reduces the area of a canal, unless the bottom and sides are uniform and smooth, it should always be installed. If there is plenty of room, a plain, unsheeted canal can be made, in earth, with good uniform bottom

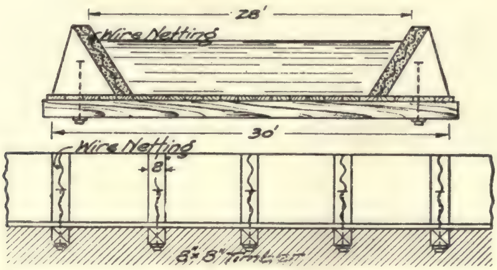


FIG. 166.

and sides and having $n = .024$. All large stone should be removed and the entire surface tamped. But where a hill rises on one side, or where a high velocity is required, it often becomes necessary to sheet the canal.

A timber sheeting is good on the bottom, the coefficient n being small for planed plank, but where the sheeting is near the water line it should be made of more permanent material. The sheeting shown in Fig. 166 was designed by the author and is a combination of timber and reinforced concrete. The timber mat here shown consists of a single platform of planed planks two inches thick, nailed to timbers placed across the canal at intervals of five or six feet. This mat is held down by the weight of the concrete but where the canal is so wide that the sills will not reach, the mat will have to be made double and filled with gravel (see page 220). The slopes are built of

concrete and are from six inches to 12 inches thick, depending on the height, the buttresses having the same thickness when wire reinforcement is used. The cost of such a lining is not at all prohibitive. The slopes for a canal six feet deep cost all complete about $3.60 per linear foot, of canal with concrete at $6.00, and netting at three cents per square foot. The bottom sheeting would require, for a canal 28 feet at the top, 65 square feet of mat and would cost two cents per square foot for laying. This would make the entire sheeting cost (lumber at $18.00) about $6.00 per linear foot of canal. If the bottom is also of concrete-steel, as in Fig. 167, the cost would be about $3.00 more than the above. To offset the cost of sheeting, we have the decreased area of canal, making it cheaper to build.

Take two canals, one having $n = .01$ and the other .03, then the sheeted canal having the same area and slope will carry fully twice as much water as the rougher canal.

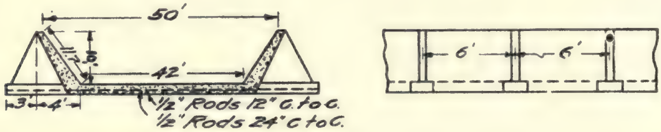


FIG. 167.

To illustrate the saving in cross-section by sheeting the following example is given:

A canal having a section like that shown in Fig 167 is built with a fall of two feet per mile.

Then

$$s = \frac{2}{5280} = .000378.$$

The area $A = 460$ square feet, the wetted perimeter $P = 65.4$ and the hydraulic radius is

$$r = \frac{460}{65.4} = 7.1$$

Taking $C = 195$ (see page 26),

$v = C\sqrt{r}\sqrt{s} = 195\sqrt{7.1}\sqrt{0.000378} = 10$ feet per second.

and

$$Q = 10 \times 460 = 4600 \text{ cubic feet per second.}$$

If the canal is in earth and must carry about the same amount of water, assuming a section shown in Fig. 168

$$r = \frac{696}{80} = 8.7$$

From formula, $C = 73$

$v = 73\sqrt{8.7} \times \sqrt{.000378} = 4.17$ feet per second.

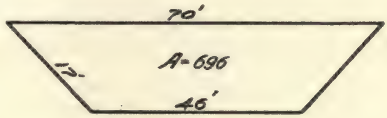


FIG. 168.

$Q = 696 \times 4.17 = 2900$ cubic feet per second, which is somewhat less than the concrete-lined canal will carry.

The fall in the two canals is the same, but the velocity head (head necessary to set the water in motion, v, at the head of canal) will be

$H = \frac{v^2}{64.32} = \frac{10^2}{64.32} = 1.56$ feet for the first section, and

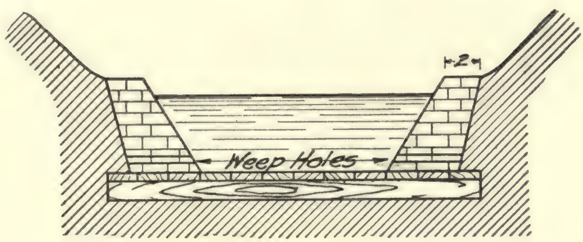


FIG. 169.

$H = \frac{4.17^2}{64.32} = 0.38$ foot for the second section.

Therefore 1.29 foot or about fifteen inches more head would be lost in the case of the canal with the higher velocity. However, this fifteen inches is for the whole canal, no matter how long.

Figuring the excavation at 12 cents per cubic yard, the first

canal would cost \$2 per foot to excavate, while the second would cost \$3.12 per foot.

The reinforced concrete section shown in Fig. 167 does not require embankments of any kind, and could be built on the surface of the ground, thus under favorable conditions saving all excavation. Also, the sheeted canal would not be choked with grass or sediment. Again, if the floor is laid tight, a much lighter embankment would be required if the section is as Fig. 168. The bottom and slopes may be paved with cobble stones or flags, and then given a four-inch coating of concrete, the proportions being about 4 to 1.

The banks may be held by means of masonry walls (Fig. 169). Unless the walls are much heavier than usually built they must be provided with weep holes, so that when the canal is emptied the water back of them will not push them in. These walls should have a good bottom when possible. If the walls are made of well-rammed concrete and the floor is tight, the weep holes are not required, in which case the sub-soil under the walls should be drained. *Masonry* walls cannot (in practice) be built water-tight. The top thickness of these walls should never be less than two feet; otherwise the frost will bulge them out after a few winters. Next to concrete, brick makes the most efficient lining.

Tunnels.

ROCK.

Frequently penstocks or canals have to be run through tunnels, as in the case of the Mill Creek plant (page 199, Penstocks). Every effort should be made to thoroughly determine the character of the materials to be penetrated. In many tunnels the estimated cost has been greatly exceeded on account of unforseen difficulties, such as caving and movement of the entire mountain above the tunnel. In selecting the section the area should be calculated with $n = .035$ and with $n = .011$. In the first case the tunnel will not have to be lined and in the second it will. Then by calculating the comparative cost, the cheaper section may be selected.

The drilling is usually done with machine drills, but may be done by hand, or both may be used. In long tunnels work is begun at both ends at the same time. The number of drills

used depends on the size of the tunnel. For a tunnel 20 by 30 feet three drills are used. Compressed air is the best motive power for the drills where the tunnels are long, as the exhausted air serves to ventilate the tunnel. A three-inch pipe will be sufficient to supply the air for six 12-horse-power drills; a $2\frac{1}{2}$-inch pipe for four drills, and a 2-inch pipe for two drills. The boiler

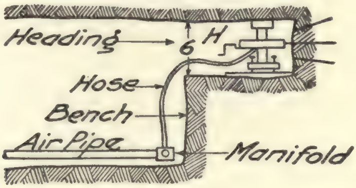
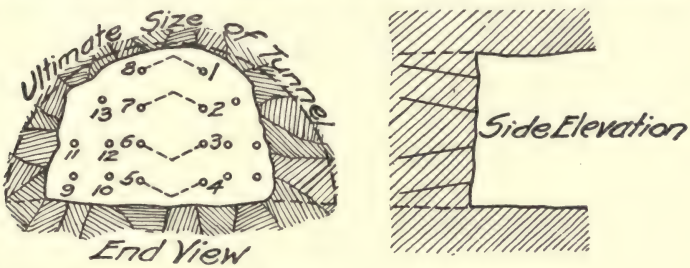


FIG. 170.

capacity should be about 13 boiler horse power per drill. The engine and compressor plant should be situated outside and near the tunnel.

In excavating the tunnel, it is divided into two parts, one called the *heading* and the other the *bench* (Fig. 170). The heading is made only sufficiently high to provide a head room for working, and is carried a hundred feet or so ahead of the



FIGS. 171, 172.

bench. In working the heading the drills are mounted on columns jacked tightly between the floor and roof, as shown in Fig. 170. The flexible hose connecting the drill to the air pipe is attached to the manifold, to which several hoses may be connected. Figs. 171-173 show how the holes are drilled. The holes 1, 2, 3, 4, 5, 6, 7, and 8, are called *center cut holes*, and, as shown in Fig. 173, each pair meets on the center line of the tunnel. The

wedged shaped mass a, b, c, (Fig. 173) is blasted out first and the operation is called *breaking the cut*. The *side round holes* 9, 10, 11, 12, etc., are drilled at the same time as the center cut

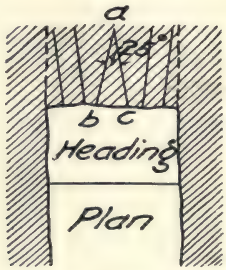


Fig. 173.

holes, and all are loaded. The center cut holes are all fired at once and then the side cut. The heading is then enlarged at the sides by drilling holes as shown in Fig. 174, the holes being

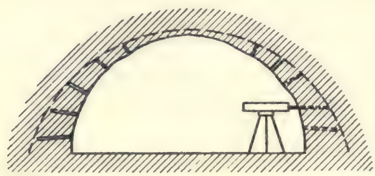


Fig. 174.

given a slant back in the direction of the completed tunnel of about 60 degrees to the center line.

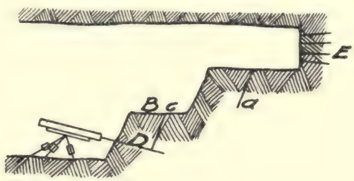


Fig. 175.

After the bottom of the heading has been widened out to the contour line of the tunnel the holes a in Fig. 175 are drilled about five feet apart across the tunnel. When a sufficient

ledge has been formed at B, more drills are started drilling the holes c. Where the drilling is perpendicular to the surface of completed tunnel as in Fig. 174 the holes are drilled to the contour, but where they are parallel to the tunnel's sides as at D and E, Fig. 175, they are drilled to within about a foot so that the explosive will not shatter the permanent walls.

The charge used must depend on the particular rock and drilling. The center cut is blasted with very strong dynamite or jovite, containing say from 50 to 80 per cent. nitroglycerine. For bench work a 40 per cent. dynamite is good. The rock should be broken up sufficiently to load into barrows without difficulty. If jovite is used for the explosive (see page 158), two pounds is used for the bench and three pounds for the center cut.

EARTH.

In tunneling through earth, especially if the earth is full of water, the engineer meets one of the severest tests of his capacity.

A brief description of the Wentworth Street sewer tunnel at Cleveland, O., will give an idea of the principle involved. This tunnel was through wet earth and under tracks which could not be allowed to settle, so the conditions were the most severe. Of course, when the tunnel is small, fewer headings have to be run and often only one is necessary.

The excavation was made in eight operations and the masonry was built in four operations. The total excavation was 21 feet high and 25 feet wide, and was made piecemeal from both sides and from the top down. First a heading seven feet high and nine feet wide, with the roof sloping both ways, was driven in the haunches of the tunnel arch on each side simultaneously. Its position and dimensions are shown in Fig. 176; it is marked 1-1, etc. After a convenient length of these headings had been driven benches about five feet deep, marked 2-2, were excavated and finally a second bench, 3-3, on each side of the center line, was excavated to below grade. After the headings 1-1 were driven two 12x12-inch longitudinal suspension timbers were laid on top of the bottom sills in each heading and supported on wedges at both ends. The sills were chained to the longitudinal timbers, and made snug with pairs of wedges. Then, bearing cleats having been nailed to the vertical posts above the sills, the bench was excavated and the timbering was sup-

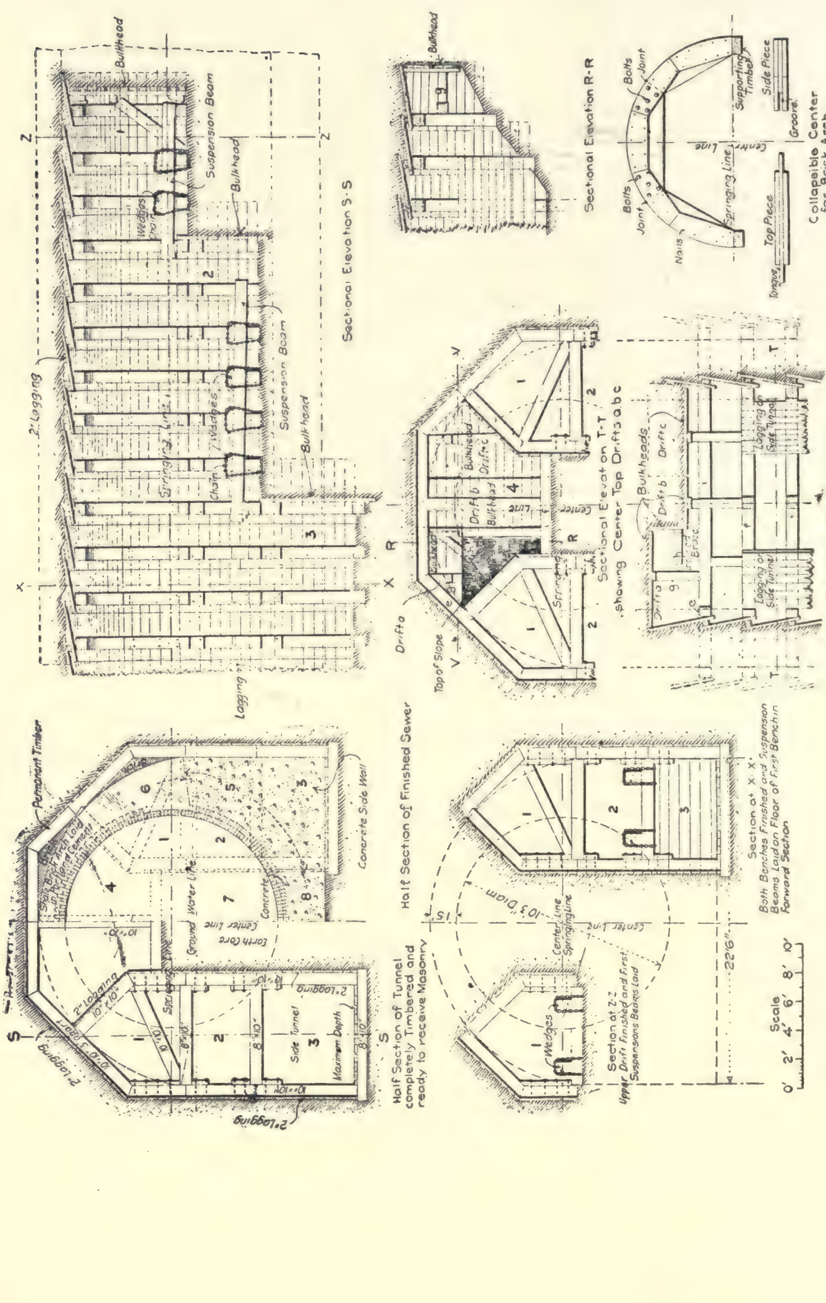

Sectional Elevation S-S
Sectional Elevation R-R
Collapsible Center for Brick Arch
Sectional Elevation on T-T showing Center Top Drifts a b c
Sectional Plan V-V
Half Section of Finished Sewer
Half Section of Tunnel completely Timbered and ready to receive Masonry
Section at X-X
Both Benches Finished and Suspension Beams Laid on Floor of First Bench in Forward Section
Section at Z-Z
Upper Drift Finished and First Suspension Beams Laid
Scale
Permanent Timber
Concrete Side Wall
Suspension Beam
Bulkhead
Wedges
Springing Line
Center Line
2" Logging
Side Tunnel
Maximum Depth
Ground Water Line
Earth Core
Concrete
10'3" Diam
22'6"

FIG. 176.

ported from the longitudinal timbers until the second sections of the vertical posts could be driven in under the bottoms of the first ones and the side lagging and lower cross struts placed. The second bench was then taken out in a similar manner.

The concrete side walls 5-5 were then built in, the vertical timbers and lagging being left permanently in the ground and the cross struts being removed as the concreting progressed. The upper center heading 4 was through very fine dry sand with about 18 feet cover and as the face had to be continually protected by vertical transverse bulkheads, it was excavated in three parallel successive drifts, as shown in the plan and sections. These drifts were started at the highest part of the tunnel and made only three feet high and three feet long, the bottom sloping downward and backward and the sides being retained by poling boards and bulkheads. The front bulkheads were braced back against temporary horizontal cross struts, and when the whole width of the heading was excavated the drifts were deepened to a point below the tops of the walls of sections 1-1 and the permanent roof timbers were put in, supported by inclined posts at the ends which rest on the apexes of the side tunnels and on vertical center posts.

Collapsible timber centers for the brick arches were made in three sections each, bolted together at the splices as shown in the elevations, with long tongue and groove joints. These centers were set between timbers on horizontal longitudinal sills supported on the concrete, and jack screws were set on top of them under the roof timbers and screwed up to relieve the pressure on the braces and allow them to be removed. The lagging was then laid as required and the arch 6-6 completed, leaving the roof timbers permanently in place, The centering was not braced or secured, but was found rigid and satisfactory.

Finally the dumpling, 7, was removed and the concrete invert 8 being laid, the tunnel section was completed. The sequence of the different operations is indicated by the numerals written on the different parts of the sections, which correspond in the several views. The material encountered was fine sandy clay and loam, about half of it being below the ground water line. The sides and roof were sheeted everywhere with 2-inch lagging in 3-foot lengths and the benches were taken out between bulkheads. The material, especially when wet, would run

very easily, and large quantities of marsh hay were used to pack in behind the lagging. It was often difficult to set the lagging and sometimes it was blocked and wedged as much as a foot back from the timbers. Wedging and cleats were used very freely and the headings were advanced about three feet in ten hours by six miners in each heading. The rest of the work was carried on at a corresponding rate and about 50 men in all were employed in each of two shifts in 24 hours.

The tunnel was driven through from one end only and materials were handled in and out of the single shaft by a boom derrick. The spoil was shoveled from bench to bench and then taken to the shaft in wheelbarrows; the concrete was mixed on the surface of the ground at the foot of the railroad embankment and wheeled to the required place in the tunnel. The bottom of the tunnel was graded about 1:1000 downward from the shaft, but all the water was removed by a steam pump with its suction in a sump three feet deep in the bottom of the shaft. The sewer was lined with very hard tough vitrified shale brick locked in to the Flemish bond of the arch rings. Notwithstanding the utmost care in tunneling the railroad tracks above the tunnel settled several inches some days, and were frequently raised and tamped with cinders. The 16-foot circular section of the outfall sewer is flattened and widened to a 50-foot channel at the outlet; it is being built in open cut through wet clay and loam, which is excavated by hand, and hoisted and back-filled by a Carson-Lidgerwood cableway of about 300 feet span.

COSTS.

Cost is such a variable quantity that only very rough figures can be given. For small tunnels of from 30 to 100 square feet section in good solid rock free from bad seams, the cost per cubic yard should not exceed \$4.00 to \$10.00 For tunnels of from 100 to 300 square feet section, \$3.75 to \$8.00 per cubic yard, and for tunnels from 300 to 600 square feet, \$3.50 to \$6.50 per cubic yard.

The cost of excavation for tunnels through earth is even a greater variable than that for rock, but taking the experience gained in several tunnels through wet earth, it might be placed as follows: Tunnels having an excavated area of 30 to 100

square feet section cost $7.00. per cubic yard, Tunnels of 100 to 300 square feet area $6.00 per cubic yard, and for tunnels of 500 square feet area and larger, $5.00 per cubic yard. These are perhaps a little bit high, but should be safe. One tunnel 30 by 40 feet cost $2.00 per cubic yard. The cost of the materials for lining is not included in either the prices for rock or earth excavation.

A tunnel of 45 sq. ft. area in hard red clay and slate cost all complete $8.28 per cu. yd., as follows:

Excavation, labor, at $2	$6.65
Blacksmith and repairs	.25
Bailing water	.17
Dynamite at 13c., caps at 3½c	.22
Coal and oil	.21
Lumber in shaling at $32	1.32
Labor in shaling at $2	.46
	$8.28

Trenches in Rock (Kidder).

Trench three feet wide in hard trap rock	Drilling	$1.50 to	$2.50	per	cubic	yard
	Explosive	.40 to	.50	"	"	"
	Throwing out	.30 to	.40	"	"	"
		$2.20 to	$3.40	"	"	"
Trench six feet to ten feet wide in Hard trap rock	Drilling	$.50 to	$.70	per	cubic	yard
	Explosive	.30 to	.40	"	"	"
	Throwing out	.25 to	.35	"	"	"
		$1.05 to	$1.45	"	"	"

Penstocks.

Penstock is the name given to that part of the headworks which *carries* the water and performs no other office. The penstock usually ends in, and becomes a part of the flume but in the flume the water brought by the penstock is transformed into power. The nearest thing to a penstock is a race or canal, and, in fact, they perform the same duty, only the penstock is built of some material other than earth.

The cheapest penstocks are built of timber. Fig. 177 shows a square timber penstock open at the top and not intended to run entirely full. In this case the cap A is wholly a tension member. The post B must be of sufficient size to resist the pressure due to the head H. This pressure will act as a point P, one-third the distance H up from the bottom end

$$W = \frac{H}{2} \times 62,5 \times H \times 1 = \text{total press against post.}$$

The size of post is then found from case (5,) (page 123)

The proper thickness of the plank will depend on the distance l, which is selected in such a way that the materials saved by thinning the plank will not add more to the frame than that saved. There is a proportion of frame and plank which is the cheapest, and this must be found by trial. Table XXXVI gives the thickness of plank for any span l and head H.

TABLE XXXVI.

PLANK UNDER WATER PRESSURE THICKNESS OF PLANK TO SAFELY SUSTAIN THE HEAD

SPAN.	40 ft. head	30 ft. head.	20 ft. head.	10 ft. head.	5 ft. head
3 feet	3½ ins	3 ins.	2¾	2¼ thick	1¾ ins
4 "	4½ "	4 "	3½	2¾ "	2¼ "
6 "	6¾ "	6 "	5¼	4¼ "	3¼ "
8 "	9 "	8 "	7	5½ "	4½ "
10 "	11¼ "	10 "	8¾	7 "	5½ "
12 "	13½ "	12¼ "	10¾	8½ "	6¾ "

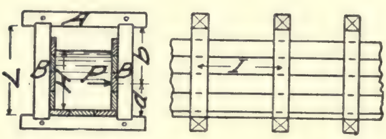


FIG. 177.

The penstock shown in Fig. 177 may be planked across the top and used full of water, and under a load higher than the cap A.

Fig. 178 shows a form of rectangular penstock which possesses some of the advantages of the stave-pipe penstock. As will be seen, it can be tightened up by means of the rods b. Such a

penstock costs more to build than the one just described, unless timber is very expensive and iron is cheap, but cheaper sheeting can be used, because of the ease with which it may be clamped up tightly. It requires more iron than a stave pipe of the same capacity and costs more to build.

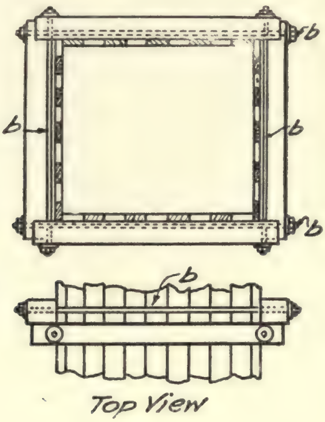


FIG. 178.

Frequently the penstock is made as in Fig. 179. In this plan the posts do not have to be as heavy as in the preceding, and there is no cap, but the braces are added and the sill lengthened.

In place of a cap or bracket t, an iron rod may be used to hold

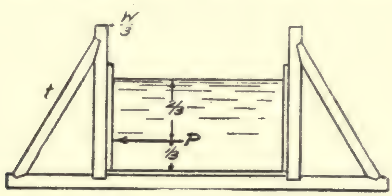


FIG. 179.

the tops of posts. The stress at tops of posts $= \frac{W}{3}$ where W is the water pressure as found above, and the horizontal stress at the foot of post $= \frac{2}{3} W$, when the penstock is planked to the

top of posts and is full of water. Of course the less the depth of water the more the proportionate stress on the foot and the less at the top.

With the growing scarcity of timber it frequently happens

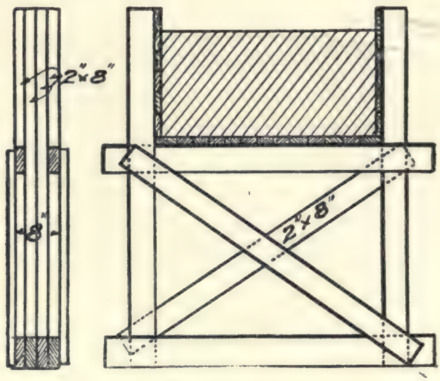


FIG. 180.

that small lumber must be used, in which case a penstock may be constructed as shown in Fig. 180. As here shown the frame is made entirely of planks two inches thick by eight inches wide. This particular penstock was made as part of the trestle and proved to be a very satisfactory one.

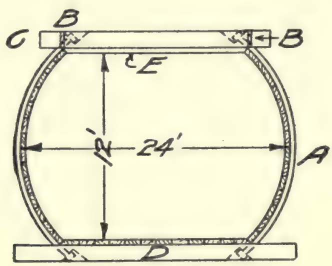
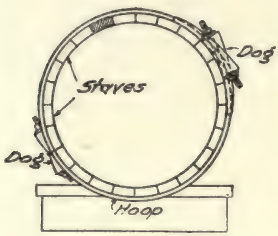


FIG. 181.

A penstock possessing certain advantages is shown in Fig. 181. The timbers *C* and *D* may cut off any segment desired. The iron bands *A* serve to tighten up the staves; *E* is a 2x8 plank, spiked on the lower end of the cap for the top stave to

butt against. Flash boards were set up along the top edges, as *B*, to hold the water up over the cap *C*, and thus prevent its rotting. This penstock is in use at Raymondville, N. Y., by the Raymondville Paper Company. The one serious defect is the certainty that the sill *D* will soon decay. If this were a reinforced concrete beam the design would be very good for an open penstock.

Penstocks built of wood staves and of circular sections (Fig. 182) are without doubt the most efficient and cheapest of all the many forms, since they possess the great advantages of permanence and tightness. Common laborers under a good foreman can construct this penstock, and at any time any part may be tightened up. The pipe does not, of course, have to run full of



FIGS. 182, 183.

water, though when it carries water under pressure it works at its greatest efficiency.

The staves are made of any good wood, but cedar, hard yellow pine, and fir are the best. Sound knots, if not more than one inch in diameter, are allowable, but there must be no sap-rot, shakes or waney edges. It is of the greatest importance to have the inside surface of the staves surfaced smooth in order to reduce the friction and increase the capacity of the pipe. The staves may be four, six or eight inches wide, and the thickness will depend on the spacing of the hoops and the pressure in the pipe. As in the case of the timber penstock, there is a certain thickness of stave and spacing of hoop which is the most economical. As this proportioning of the parts depends on the cost of the materials we will not attempt to give any data on the subject.

The usual joint at the end of staves is shown in Fig. 183. The

ends are sawed square and a kirf is sawed in with a cross-cut saw. A template should be used in sawing the kirfs in the ends of staves, as shown in Fig. 184. The fillet is just thick enough to allow the saw to enter between the hard wood guides. With such an appliance two men can easily keep a pipe gang of four men supplied. An iron tongue $1\frac{1}{2}$ inches by width of stave $+\frac{1}{8}$ inch, and $\frac{1}{8}$ inch thick is slipped into the slot after the stave

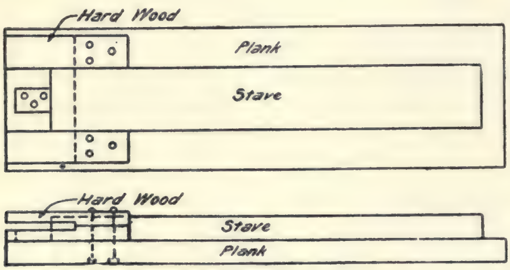
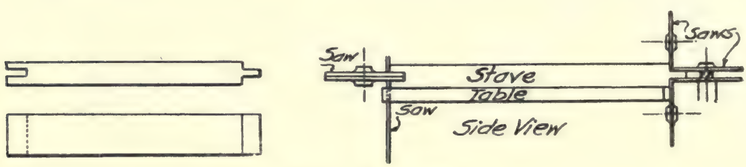


FIG. 184.

has been put into the pipe, as shown in Fig. 183. Being $\frac{1}{8}$ inch longer than the space the ends crush into the staves and form a water-tight joint. These tongues *should be galvanized*, otherwise they will rust out in the course of four or five years.

The opinion of the author is that the best stave joint is the



FIGS. 185, 186.

one shown in Fig. 185. In this joint there is no iron to rust, and the tongue being of wood and exactly the width of the stave, will swell tight. To saw these ends requires a special saw, something like that shown in Fig. 186. Of course such a sawing outfit is expensive, the one shown costing \$150, but on long pipe lines the steel tongues saved will more than pay for the cost. With such a sawing outfit a stave can be sawed in much less time than the ends shown in Fig. 183 could be sawed by hand. The

pipe line for which the above saw was made was composed of 45,000 staves or 90,000 ends. A saving of 18,000 pounds of tongue iron was affected, which, with iron at three cents a

Fig. 187.—Stave penstock.

pound, saved $540, and fully as much again was saved on the sawing.

In building the pipe a tool called an expander is used to

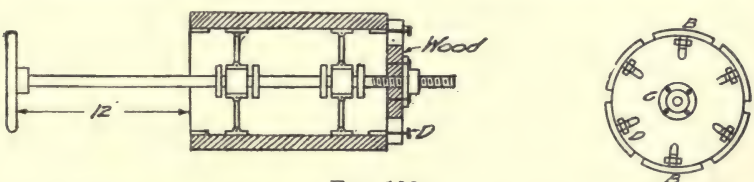


Fig. 188.

keep the pipe a true circle. This is shown in Figs. 187 and 188. The expander is kept out to the full diameter while the hoops are being placed, as the rods are tightened the expander is contracted by turning the handle.

Fig. 188 shows the end of expander. In the end of each

segment B is an iron D which works in a slot in the head C. The expander shown will do for pipes up to four feet in diameter, but beyond that men can work inside the pipe and a simple device like Fig. 189 is used. It consists of a disc of wood whose diameter is exactly the size of the pipe, and having the two segments mounted on hinges as shown. When the hoops have been tightened some, the hinged parts are knocked back and the disc moved along. A manhole is provided so that workmen can pass through.

Round iron is usually used for hoops though flat iron is sometimes best for very heavy work. Round iron crushes into the wood some and thus relieves the pressure caused by the swelling of the wood. It is also ready to thread and may be cut to any length. A hoop smaller than ½-inch-round iron should never be used on account of the effects of rapid rusting. To

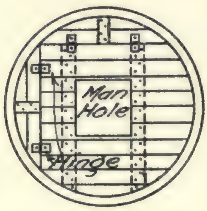


Fig. 189.

allow for the rusting away and the swelling of the wood, a factor of safety of eight should be used. Fig. 190 will be found useful in getting the proper size of hoop. The left hand ordinate gives the pressure in pounds per linear inch of pipe.

Example.—A pipe 10 feet in diameter under 500 feet head has a pressure of 13,000 pounds per inch length, tending to part the two halves of the pipe and, if a rod is used every inch of the pipe's length to hold the pressure of 500 feet head, it will have to have a safe strength of 13,000 pounds. If the rods are spaced six inches apart, each rod will have to have a safe strength of $13{,}000 \times 6$, or 78,000 pounds. If the pipe is made of metal, the material must be such as to safely resist the pressure of 13,000 pounds per inch length. Assume the metal is ½-inch thick, it must then have a safe tensile strength of 26,000 pounds per square inch. In this way, the table may

be used for spacing of hoops and for any kind of metal pipe. The diameter of the rod should be taken at bottom of the threads.

The shoes or dogs, are usually of cast iron and made separate from the hoop. The author has found the shoe shown in

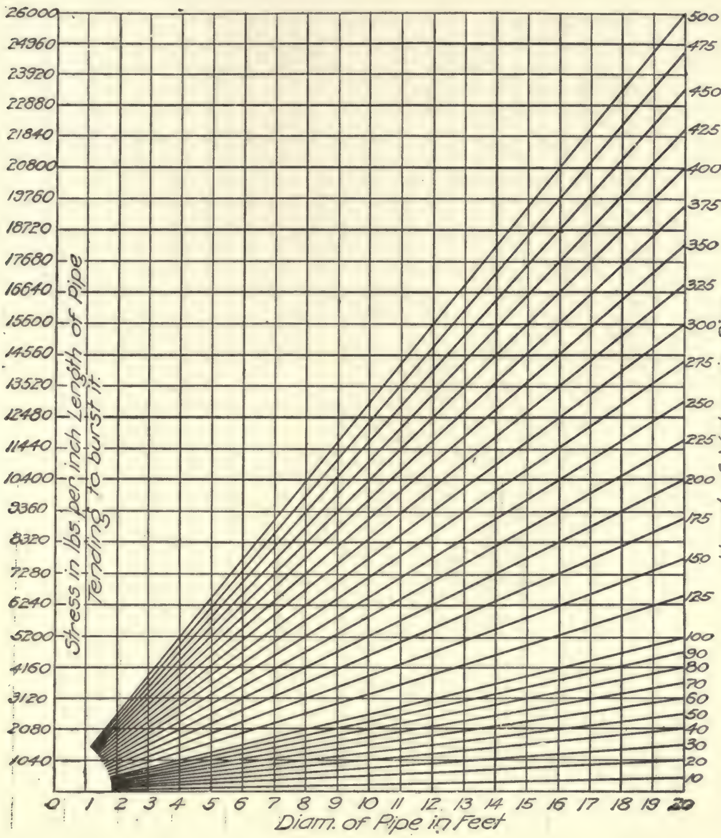


FIG. 190.

Fig. 191 to be the cheapest and best to use. This one is designed for a $\frac{5}{8}$-inch rod. Its advantages are that the hoop does not have to be strung through a round hole but simply bent over into it. On jobs of any size it is a good plan to make socket wrenches with auger handles for tightening up the hoops.

COSTS.

Fig. 192 gives the cost per foot of reinforced concrete penstock. The data from which this curve is plotted was derived largely from Gillette's book, "Cost Data," but also from numerous other sources.

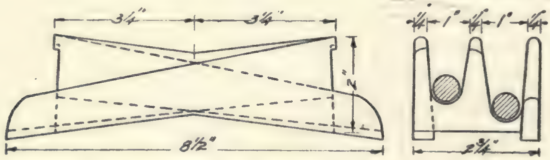


FIG. 191.

Reinforcing costs about three cents per pound for steel, and 0.5 cent to instal. The concrete costs bout $10.00 per cubic yard, including every item. Round rods cost about

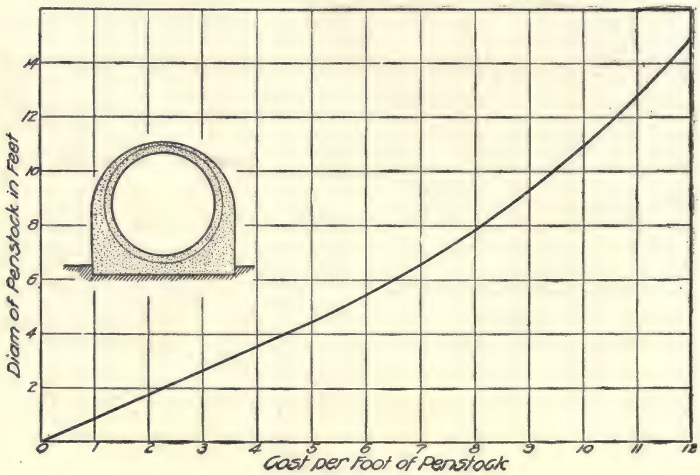


FIG. 192.

$34.00 per ton. Brick penstocks require 570 bricks per cubic yard and 1.25 barrels of cement. A mason should lay 1200 bricks in eight hours at a cost of $6.00

Figs. 193 to 195 show the three common forms of steel riveted

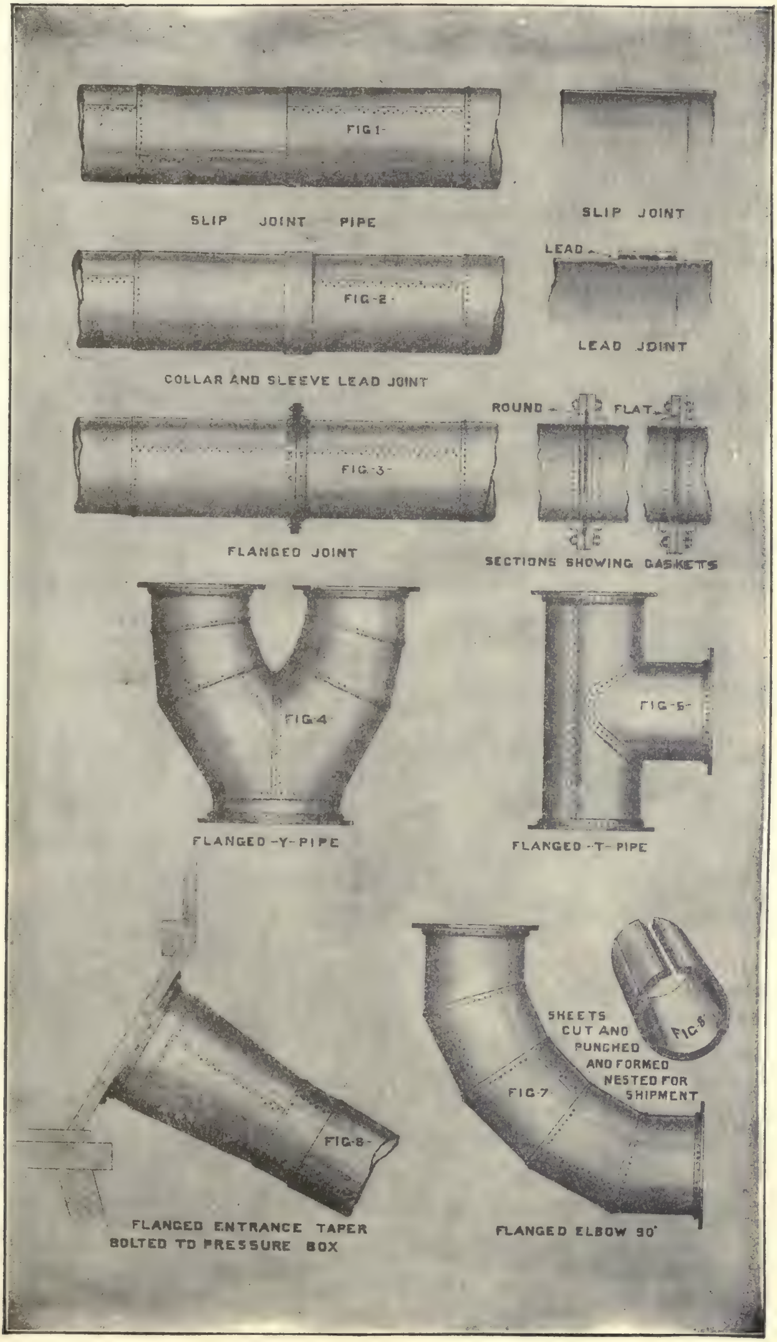


FIG 1
SLIP JOINT PIPE
SLIP JOINT
LEAD
FIG 2
LEAD JOINT
COLLAR AND SLEEVE LEAD JOINT
ROUND
FLAT
FIG 3
FLANGED JOINT
SECTIONS SHOWING GASKETS
FIG 4
FLANGED -Y- PIPE
FIG 5
FLANGED -T- PIPE
SHEETS CUT AND PUNCHED AND FORMED NESTED FOR SHIPMENT
FIG 6
FIG 7
FIG 8
FLANGED ENTRANCE TAPER BOLTED TO PRESSURE BOX
FLANGED ELBOW 90°

Figs. 193–195.

pipe. Fig. 193 shows the common *slip joint*, one end being tapered so as to drive tightly into the preceding section. The fit of the ends is alone depended on to secure a water-tight connection, no riveting being done. Such joints should not be used for heads exceeding 250 feet.

Fig. 194 shows a butt joint, leaded. In this type of joint the ends butt squarely against each other. A sheet iron sleeve is riveted on the inside of a section as shown and the next length placed over the projecting one. A steel collar about ¾-inch larger inside diameter, than the outside diameter of pipe, is then placed around the joint and run full of lead, and securely caulked. This joint can stand a head of 600 feet.

The flanged joint, Fig. 195, is perhaps, the best joint for heads above 600 feet, and can be made for any pressure. The figure is self-explanatory.

The most common point of failure is the breaking of the flange and not the stripping of the thread. The end of the pipe should be screwed an eighth of an inch beyond the face of the flange so that it will press against the packing. The faces of the flanges should be planed smooth and corrugated male and female. Rainbow or copper gaskets are used between the flanges.

There has long been a strong prejudice against the use of thin steel for water-pipes on account of the rapidity with which they rust out. The Pelton Water Wheel Company of San Francisco claim that they greatly retard the rusting process by boiling the sheet steel in hot asphaltum. A great advantage possessed by this pipe is its lightness, making transportation easy. Long straight steel pipes must have expansion joints.

The Designs of Dams.

MATS ON SOFT BOTTOMS.

Under this heading all bottoms other than solid stone are included. A great deal of care must be taken in selecting a good bottom.

The most common river bed is one having large round stones on top of gravel, and the gravel covering a stratum of clay or sand, usually the latter.

Frequently this layer of stone and gravel is quite thin, being merely a film covering the most dangerous sand. An iron rod

can hardly be driven into such coverings, and hence often leads the engineer to think that the bottom is a good one.

Dams on such bottoms must be placed on some form of mat, and at least one row of sheet piling must be driven. Fig. 196 shows a mat under construction. Here the gravel and boulder covering was part way directly upon the clay, and part way over very elusive sand. Sheet piling could not be driven through this layer, so trenches were dug as shown.

Hard pan is a name given to a hard clay mixed with small

FIG. 196.

stone. It is very hard and has to be picked or blasted. Bottoms of hard pan, provided the hard pan is thick enough, may be built upon direct, without the use of a mat, piling cannot be driven, so a seepage trench similar to the one shown in Fig. 237 is used to prevent seepage underneath. The hard pan may be overlaid with some softer material, as sand or gravel, in which case great caution must be observed. Where possible the soft bottom should be excavated and the dam placed on the hard pan. Where the

depth is too great to make this possible, sheet piling must be driven. Where the hard pan is only three or four feet from the surface, there is danger that the seepage confined within narrow limits will cause undermining. Under such conditions the sheet piling must be even more nearly water-tight than would be necessary were the hard pan ten or more feet down.

When the bottom is of sand every precaution known to the profession should be used. When contained, sand makes a perfect foundation, but if allowed ever so small an outlet it will run like so much oil. There is a great difference in river sand with regard to difficulty in building. A sharp coarse sand is safety itself compared with some of the fine dull sand found in the Western rivers.

On sand bottoms the width of the mat should be about four times the height of the dam, and a very heavy extension mat

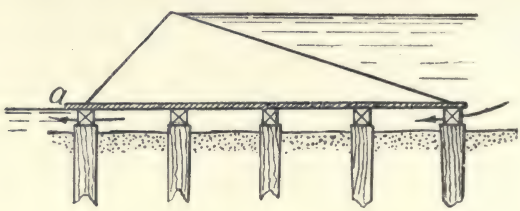


Fig. 197.

must be provided. A row of water-tight sheet piling must be driven along each edge of the mat. These should be *jetted* down, unless they are of steel. On the worst bottoms the piling should be from 18 feet to 100 feet long, depending on the height of dam. Nothing must be placed under the mat to hold it up, other than the sand, as the entire weight of the dam and water is required to press the mud sills into the sand. Many dams have been undermined because they were placed upon piling. The sand settled as in Fig. 197, and though sheet piling was driven, enough wat seeped through to cause a washout.

There is always more or less seepage through sand. The voids are full of water and water pressure at one point is instantly transmitted to more distant points. Thus if the mat is laid upon the sand and more water seeps through the up-stream sheet piling than through the down-stream row, there

will be an uplift on the under side of the entire mat. Therefore, outlets must be provided through the top of the mat. Usually where the worst sand is found there will be no stone for riprapping and concrete, but gumbo and sand are apt to abound, and with these rock may be created.

MATS ON PART ROCK AND PART SOFT BOTTOM.

A condition sometimes met with is as in Fig. 198. Here the bottom is rock part way and sand the rest of the way. Where the rock drops down, as at *a*, a wall is built for the mat to abut against. The mat has no connection with this wall, as it would be held up from pressing the sand underneath and a leak would follow the wall. This wall is built up above the

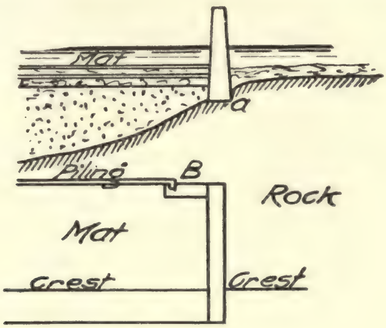


FIG. 198.

crest of the dam. At *B* the wall is carried out toward soft bottom and down to rock, to a depth of six or eight feet or more, and the first sheet pile imbedded in it as at *B*. This shuts off all leakage and yet does not suspend the end of the dam over the soft bottom. The large dam at Lowell, Mass. is an interesting example of a dam built partly on rock and partly on soft bottom. It has been a constant source of expense on account of the method of building.

MATS ON ROCK BOTTOMS.

Dams built upon rock are not necessarily built on a safe foundation. To be safe the rock must be free from seams which would conduct water under the dam. The Austin dam was supposed to be built on a solid ledge, but water went under the dam and came out far below. Deep sounding with a core

drill (page 57), is the only way to make sure; but for small dams a seepage trench, see Fig. 237 will be sufficient, and in the excavation, show up the nature of the rock. The proportions of the seepage trench will depend on the nature of the bottom and whether the safety of the dam depends solely on the gravity of the masonry or upon the gravity of the impounded water.

If the former it should be wide enough, up and down stream, to afford the proper shearing strength (unless the entire dam is set down into the rock as in Fig. 237, to resist the total down-stream thrust of the water. If the dam is of the gravity type the trench need only be wide enough to prevent water seeping through.

On bottoms that are not too yielding—such as sand and silt—the mat may be made of concrete and steel, and at a cost

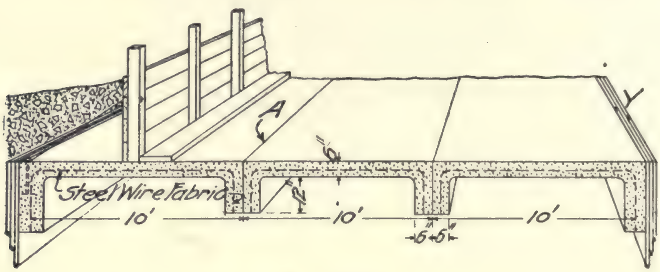


FIG. 199.

of about the same as timber when concrete can be made for $5.00 per cubic yard and timber costs $30.00 per thousand.

Such a mat is shown in Fig. 199. The reinforcing consists of $\frac{5}{8}$ steel rods. At the points between sections, as at *A*, thin tar-paper is placed during the laying of the concrete, so that in settling the mat will not break into irregular cracks.

The cost of this mat laid on bottom fairly level would be about 20 cents per square foot. The rods will cost $1\frac{1}{2}$ cents per square foot, and the concrete should easily be laid for $5.00 per cubic yard. In all foundation work it is very important to work in the dry, and the time saved and superior excellence of the work will pay for a liberal expenditure for coffer dams and pumping.

EXTENSION MAT.

There are very few bottoms where a mat is required, which do not also demand the use of an extension to conduct the water away from the dam in safety.

One of the most common methods is to build a heavy crib below the mat and fill it with stone (see Fig. 200). The crib is not attached to the mat, but is held in position by heavy

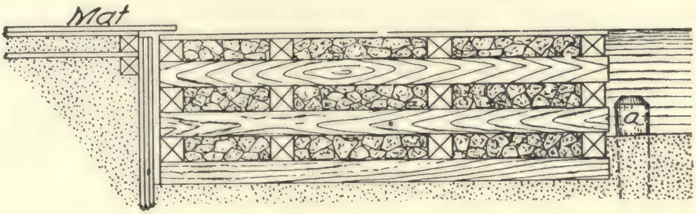


FIG. 200.

round piles *a*, driven at six-foot to ten-foot intervals. This allows the crib to settle without pulling away from the mat. This crib should take all the pounding there may be, and thus save the dam from the vibration. The deck should be of oak from four to six inches thick. The proportions of the crib will depend entirely on the nature of the bottom, but it is usually

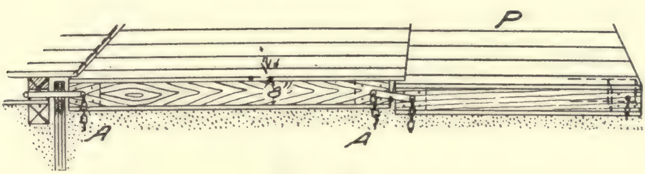


FIG. 201.

best to get the desired degree of safety from the undermining by making the crib long rather than deep.

Another form of mat which possesses many good points is shown in Fig. 201. This consists of a series of wooden boxes, having plugs in the ends, and filled with sand. Each box is strung on two chains, as at *A*, running across the river. If a longer extension than 16 or 18 feet is required, another section may be attached, as shown. The extension is fastened to the

mat with links passing through the piling, and the intermediate sill. Such a mat is perfectly flexible, and will continue to settle as long as there is any wash.

About the cheapest kind of an extension mat is that shown in Fig. 202. It is hardly a safe construction where the bottom is sand, as the back wash works back under the flooring an incredible distance. The pilings must be driven deep, as they often have to resist severe uplifts from ice, and this at a time

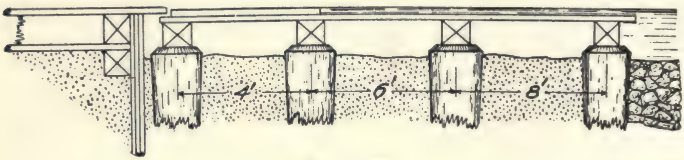
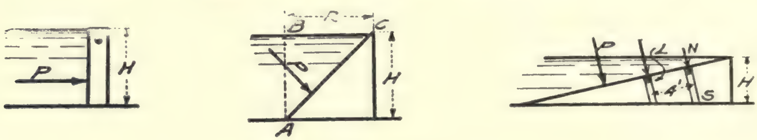


FIG. 202.

when the earth may be washed away for half their length or more. The piles should be driven at intervals of from six to eight feet across the river.

GRAVITY DAMS.

The gravity dam is one of the oldest types, but it is only within the last few years that it has become a prominent type of construction. To more clearly understand the theory of the gravity dam we will investigate the action of the water



FIGS. 204, 205, 206.

in the three cases (Figs. 204-206), Fig. 204: here the water acts at P and is wholly a horizontal force tending to shove the dam down stream. This pressure can be figured with great exactness, and $= \frac{1}{2} H \times H \times$ the weight of a cubic foot of water, the only factor of uncertainty being the weight of water and the extreme of variation for all altitudes and temperatures is only 0.5 per cent., 62.41 pounds per cubic foot being the maximum weight; Fig. 205: in the case of a dam having a slope up stream of 45°, we have the same down-stream pressure as in Fig. 204

(H being the same), but we also have a vertical pressure. Suppose $H = 20$ then the horizontal $P = \frac{20}{2} \times 20 \times 62.5 = 12500$ pounds per foot length of dam. We have a vertical pressure equal to the weight of the triangle of water, $A\ B\ C$, immediately over the dam $= \frac{H}{2} \times R \times 62.5$. As $R = H$ in this case, the horizontal and vertical pressures are equal. If there were no friction between the dam and the bed of the stream the dam would be just ready to slide. Fig. 206: here we have an exaggerated gravity dam where practically all the pressure is vertical, and at any point N the pressure is perpendicular to the deck and almost perpendicular to the base of the dam. The horizontal pressure on the dam depends solely on the depth of water, and is the same for all forms, but the vertical pressure for equal depths of water may be made to assume any desired value by changing the form of the dam. This pressure is found by multiplying the depth of water above N by 62.5, which gives the pressure per square foot on the deck. Suppose a post S is used to resist this pressure and the distance between posts across the stream (distance between bents) is four feet, and the distance up and down stream between lines of posts is four feet, then the posts will sustain an area of deck equal to four times four, or 16 square feet. This multiplied by the pressure per square foot gives the pressure on the post, and the post may then be designed as exactly as can a compression member in a bridge, and even more so, because the load is a constant one. If the water stands 10 feet above the crest of the dam 625 pounds of pressure are added to each square foot of the deck's surface. The deck is usually attached to plates, as at L, in which case the span between posts (in this case four feet), and the plates being four feet apart, each plate sustains 16 square feet of the deck, or 16 times the pressure per square foot on the deck. Therefore we have a beam four feet long carrying a uniform load, and from tables we find the proper size of beam, using any factor of safety we may choose. Here all uncertain factors are eliminated and the design of the dam becomes simply a question of strength of materials. The posts are placed at right angles to the deck and in direct line with the thrust of the water.

The flatter the dam the greater the pressure down upon the river bed; therefore for dams on sand bottoms the slope should not be greater than 30°; 23° is about as flat as necessary for the softest bottom, and for dams on rock bottoms 45° may be used.

One of the oldest types of the gravity dam is the crib dam. Usually it was filled with stones and earth. Fig. 207 shows

FIG. 207.—Common form of crib dam.

such a dam, built entirely of 4x8-inch timbers, all spiked together to form numerous cells, each about six feet square. In this particular dam the cells were filled with gravelly dirt, thoroughly wet down. This is a fair type of the crib dam. Frequently logs are used instead of the 4x8-inch timbers, and the dam filled with stone. In nearly every case where the bottom is soft, the dam is supported on piling. A dam is built to utilize

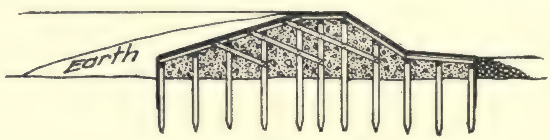


FIG. 208.—Pile dam.

the holding-down force of the water, and then expensive piling is driven to hold the dam up off the bottom.

In this dam there is no means of entering the interior, so that the timbers may be inspected and renewed at will. Filling with earth or even stone hastens decay and adds nothing to the security of the structure. Such dams are usually filled above with earth (Fig. 208), gravel and boulders. Boulders make the very worst

filling possible when it is desired to stop seepage. In fact they make a drain rather than a stop-water. A common form of crib gravity dam for soft bottoms is shown in Fig. 209, and for rock in Fig. 210. All the cribs are filled with stone. The timbers are mostly 10x10 inches, and the apron is of 12x12-inch timbers. The apron is shown without the planking. This dam

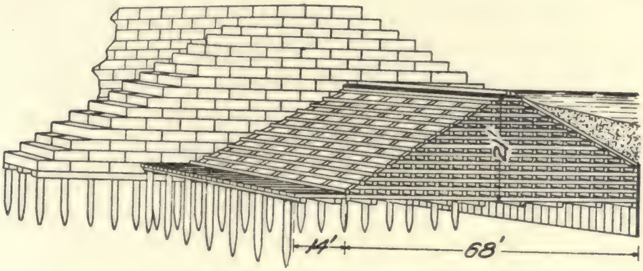


FIG. 209.—Crib dam.

stood for about 50 years. The bottom was boulders, gravel and sand. This is a very good construction, the chief criticism being that too much material is used, that it cannot be entered for inspection, and that decay is increased by such masses of heavy timber.

Every timber entering into the construction of a dam should

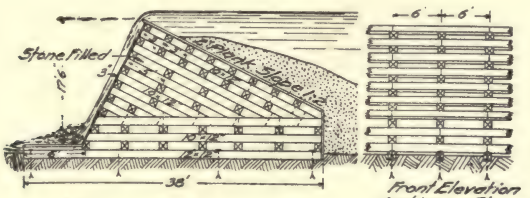


FIG. 210.—Crib dam.

be proportioned to the load it is to sustain. This is for two reasons, cost and longevity. It is a well-known fact that a large timber will rot more quickly than will a small one. In a large stick the heart decays first, and while the outside may seem to be sound, the interior becomes soft and devoid of strength. Every timber should be accessible for inspection and repair. The interior of the dam should be ventilated to keep down the

temperature, which is at all times higher than the outside air. A large portion of the timber should at all times be in direct contact with the water.

The dams shown in Figs. 207 and 209 are resting on mats of solid timber and bolted thereto. In Fig. 207 the mat is supported on piling, but in Fig. 209 it rests on the river bottom. The mat is an essential feature of all dams on soft bottoms, and should possess the following features: It should extend under the entire dam and the abutments. It should have a specific gravity heavier than water. It should be flexible. All of the sills in contact with the bottom must run *across* the stream and not up and down stream. The mat must *not* be supported on piling. It must have at least one row of sheet piling along the edges.

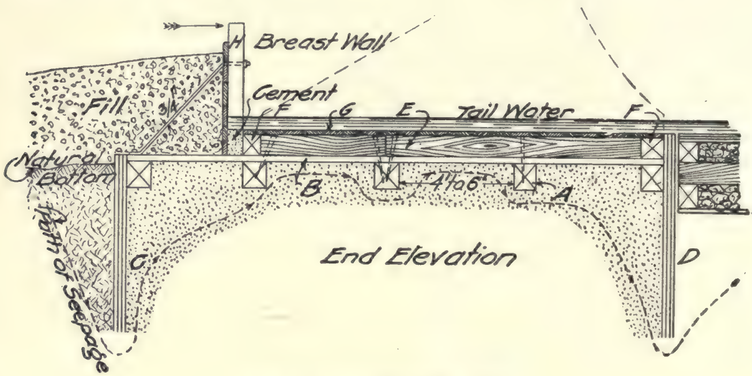


FIG. 211.—Beardsley mat.

In Fig. 211 the essential features of a patent mat possessing all the above features are given. The mat is built so that it is at all times submerged. It can therefore be built of any sound lumber. The mud sills A are first laid in the river bed. Where possible the river bed should be excavated rather than filled up to a level. That is, the top surfaces of the mud sills should be placed level with the natural river bed, the trenches being dug for them. When this cannot be done the fill may be made with any material that will not be dissolved by the action of the water. An architect's level is indispensable in laying these sills. One man holds the staff on the ends of the sills and the other men tamp under the sills until they are level, then the timbers are

weighed down to prevent floating out should the pumps stop. When the sills are levelled they are filled flush to their tops, a straight edge being used to level with.

If too much fill is used the nailing of the plank will draw the sills up, and if too little there will be uneven settling. Next, the planking *B* is nailed down. Unless these planks are edged and well dried they should be battened water tight on that part of the mat over which the water will be conducted during the building of the last half. It is a good plan to make this deck of two layers of 1-inch boards, breaking joints at edges and ends. This makes the mat more flexible. The row of sheet piling *C* is driven as deep as possible and a water-tight connection made with the mat. The row *D* is not attached to the mat, and the spaces usually left will permit the seepage water to find an outlet without exerting an uplift on the mat. It is also desirable to permit the mat to settle, which it could not do if it were fastened to the piling. The intermediate sills *E* run up and down stream, being placed a distance apart equal to the distance between the dam bents, so that each bent will rest directly over a sill. The sills *F* run lengthwise of the dam, thus forming, with the sills *E*, a series of compartments, each as wide as the distance between dam bents, and as long as the mat. These compartments are filled with gravel or stone, and the top planks *G* are then laid. The top planks are merely for the purpose of conducting the water over the mat without allowing it to wash out the gravel, so they need not be edged or the cracks battened. Along the up-stream edge of the mat is built the breast wall. The posts *H* are placed five or six feet apart and should be from four to ten feet in height, depending on the height of dam. The tops should not come near enough to the surface to be struck by floating ice, logs, etc. The breast wall serves two very important purposes: During construction it is used to shift the water from one part of the mat to another, so as to aid in building the dam. By its use the water may be raised, a plank at a time, until the fill above it is completed. It also serves to hold the fill over the edge of the mat, the only place where the water could possibly find an outlet. This fill is made as high as the current going over the breast wall will permit. The breast wall is not placed on top of the plank *G*, but on the lower course. In this way greater flexibility is obtained between the mat directly under the dam and where it

is attached to the piling. Also a small amount of fill can be made over the edge of the mat without filling above the top level. The posts are held by means of rods, or may be braced. The timbers used for heads up to 20 feet are 8x8 inches, and for very low heads 6x8 inches.

The length of the mat should be such that the foot boards will not strike the breast wall in being dropped, and that the down stream edge of apron will just come to the edge. Water from the overpour should never be allowed to strike the mat. It will be seen that it is a physical impossibility for water to ever cut under such a mat. In an experience with over 60 such mats the author has never known the water to get under one. Owing to cheap construction several have been undermined from below, but never injured along the breast wall.

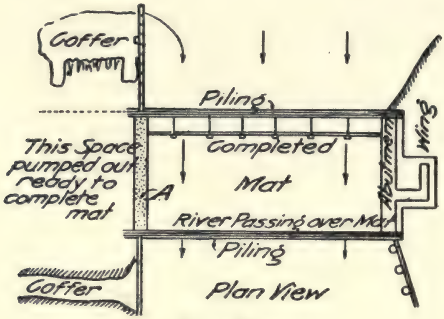


FIG. 212.

Fig. 196, gives a good idea of the construction of such mats. In this case the water was all turned through the power house, seen in the distance, and the breast wall was filled full depth at once. The trench shown to the left was to make the driving of the sheet piling more easy. Each alternate sill should be drifted to the mud sills with at least 2—$\frac{5}{8}$x14-inch drift spikes. The planking is done with 30*d* spikes.

In the majority of cases it is necessary to build half of the mat at a time. During the building of the first half (or as much more than half as possible) the water is turned to the other side of the river, but while building the last half it runs over the completed portion. (See Fig. 212.)

In this case the entire surface of the mat is exposed to the

water, and if there is any leakage it will follow along the sills toward the uncompleted mat, when the water is pumped out, unless a cut-off wall, *A*, is put in. It will pay to do a good job on this cut-off. The author has found that a concrete wall as shown at *A* Figs. 212 and 213 is the best. This wall is built before the mat is laid near to it on either side, and should go down to firm bottom if possible. The top must not be more than an inch or so above the top of the mat, and only extend to the up and down-stream row of piling; along the top is embedded a plank or timber to which temporary planks may be nailed. These temporary planks are tongued and grooved and keep out the water. The mat simply abuts against the wall and does not project into it at any place. A row of piling may be used in place of the wall.

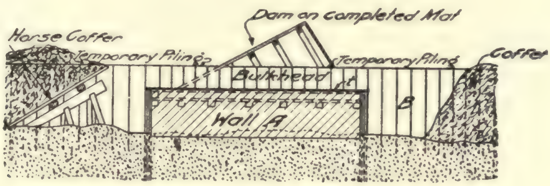


FIG. 213.

The cost of laying a mat is about 3 cents per square foot, including digging the trenches for mud sills, leveling, planking, etc., but does not include the fill above breast wall. Three kegs of 30d nails are required to 1000 square feet of mat. A timber mat requires from 5½ to 7 square feet of lumber per superficial square foot.

The dam and the abutments are placed on the mat. The dam may be of the crib type if desired, though commonly the frame dam is used. It may, of course, be fastened down to the mat by means of drifts, but this is wholly unnecessary, as once the water pressure is on nothing could stir the dam. A few years ago, to demonstrate to an incredulous city board that the dam would not slip off the mat, a model dam was built and placed on the slimy floor of an old penstock. The dam was four feet high and just fitted into the penstock, without quite touching at the ends. The water was turned on all at once and the

model only slid one inch and then settled solidly upon the floor.

Figs. 214 and 215 show dams of the frame type and are suited either to place on mats or solid rock. There should be no mortise and tenon joints about a dam, as experience has proved that such joints are the first places to decay. A plain sawed butt joint is all that is necessary, there being no side strains at all. Four 40d spikes are used at each joint to hold the parts in place.

The designing of a frame gravity dam is a very simple operation. Take the example of a 20-foot dam with 5 feet of water going over it. (Fig. 215.) First, assume the slant of the deck to be 23°, and draw the decking. Then at 5-foot intervals erect verticals to get the depth or head of water at those points. Multiply the depth by 62.5 to obtain the pressure per square foot on the deck. Thus, at vertical (1), when the depth is 22 feet the pressure is 1375 pounds per square foot. Now the up-stream plate *A* must, in most cases, be high enough above the mat to allow the passage of the water underneath. This makes the length of the foot boards *B* about six feet, and the plate *A* will then have to sustain practically half the weight on these foot boards. If the distance between bents is four feet, $1469\times3\times4$ is the part of the pressure on the foot boards held by the plate. It will also hold half the weight on the 3-foot span to the next plate, which is $1375\times1.5\times4$. The sum of the two is 25,876 pounds. From table (55*) an oak beam one inch thick and 10 inches deep will safely sustain a load of 2640 pounds, therefore a 10x10-inch beam will sustain 26,400 pounds. At depth (2) the pressure per square foot is 1250 pounds and the area supported by the plate *C* is $4\frac{1}{4}\times4 = 17$ square feet; therefore, the load on *C* is $4\frac{1}{4}\times4\times1250=21{,}250$ pounds, for which an 8x10-inch timber is found to be right. As it is not best to use a smaller timber than an 8x8-inch in a dam of this size, all the remaining plates will be made of 8x8-inch timbers, and they should be spaced up and down stream so that the full strength of the decking will be utilized. At (3) they could safely be four feet apart, and at (7) six feet, so six feet will be taken as the maximum distance at the crest and then gradually diminish the span toward the toe.

The posts, if made of 8x8-inch timbers, will be many times stronger than necessary, but a smaller size would make the pressure at the ends too severe. From table (54*) a post six

* See Chapter IX.

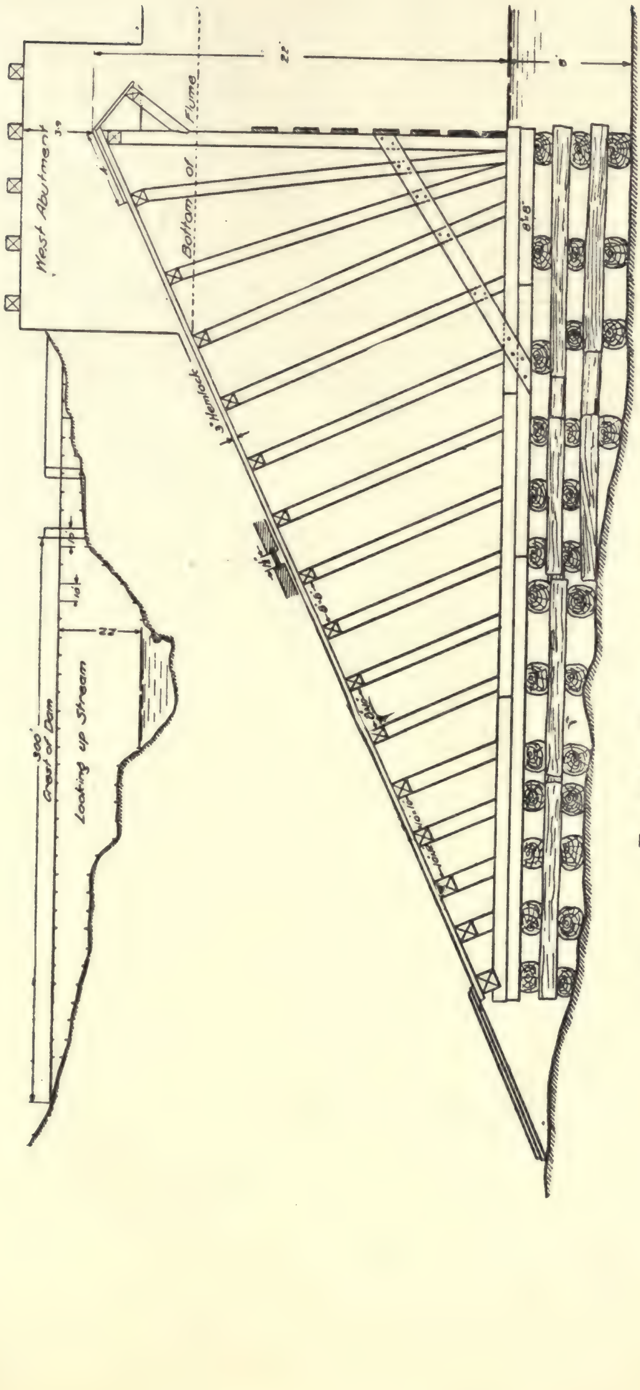


FIG. 214.—Timber gravity dam, rock bottom.

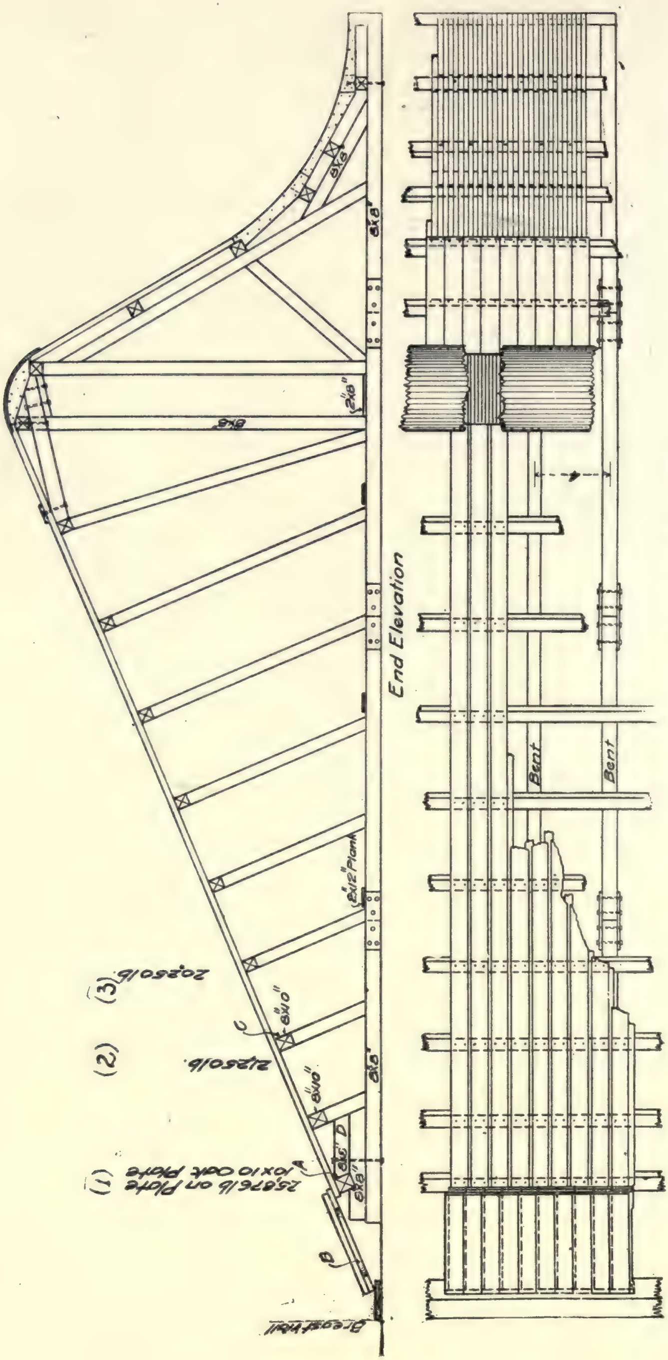


FIG. 215.—Timber gravity dam, soft bottom.

feet long and 8x8 inches section will safely sustain 41,000 pounds for white pine, and for oak about 52,000 pounds. At *D* the lock block is bolted to the sill with a ¾-inch bolt. This block stiffens the dam against the horizontal forces acting at crest of dam, such as ice expansion.

The design of the apron is more important perhaps, than any other part of the dam. If the water is given no object to strike against the only way the apron can be injured is by wearing out under the friction of the water. In this design a curved apron, having a crest formed to prevent the water from falling perpendicularly over on to it, is shown. This crest can be covered with boiler iron. The curve is obtained by nailing together segments made of 3-inch plank. The straight part of the apron is made of 3-inch white oak or yellow pine, and the lower portion is built up like the crest, the segments all being sawed to template before placing on the frame of the dam. The only strain on the timbers is that due to the weight of the water, and when passing over the apron this is in a very thin sheet. The segments of the apron should be sawed so that the grain will run with the current.

The foot-boards *B*, are of 4-inch plank. Each bottom board is notched on one edge so that it will bear its part of the pressure. If the bottom boards were water-tight no pressure would come on to the top boards at all. Curve (Fig. 258) gives the thousands of feet of lumber in 100 feet of dam similar to that just designed.

Fig. 216 shows a gravity dam, made principally of steel. The details of construction may be worked out in a great many different ways, but the design shown will serve to illustrate the principle. The deck is of tongue and grooved plank, as in the timber dam. As the deck is at all times in direct contact with the water it is preserved from decay. The apron is made of segments as shown. If, for any reason, the sill cannot be placed under water, another channel should be used in its place, forming, with the one shown, a box girder.

Fig. 220 shows a design for a concrete-steel dam. This form of dam is the combination of all that is good in both the timber gravity dam and the concrete dam. The use of the steel reinforcing makes the design as certain as it would be for an all-steel dam. The compressive strength of the concrete is used

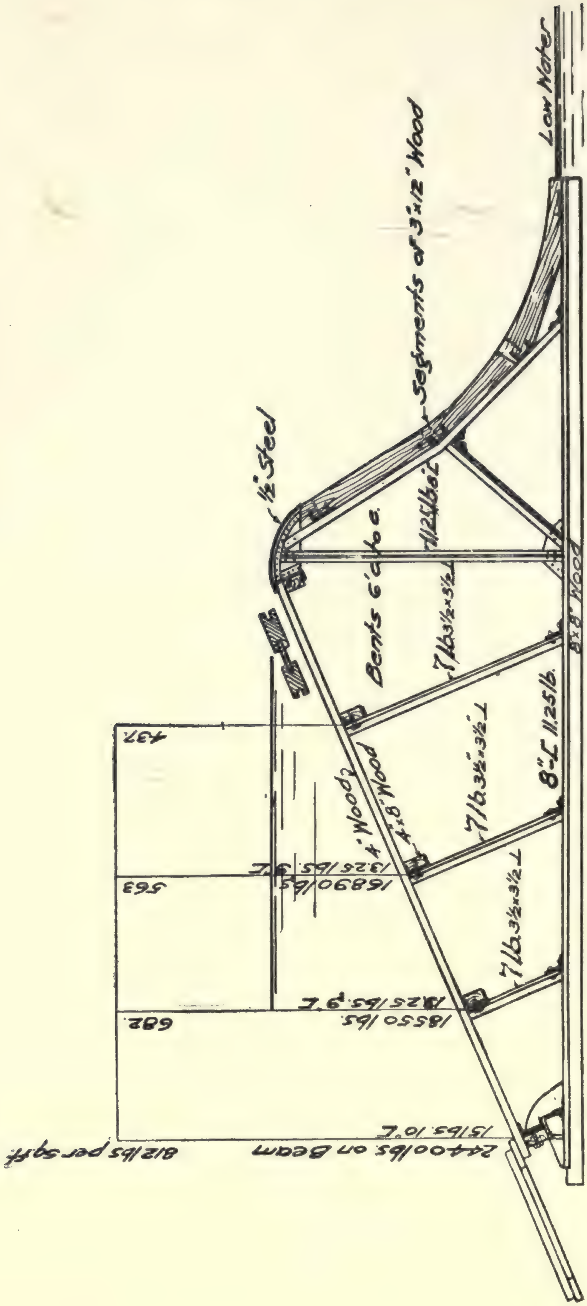


FIG. 216.—Steel gravity dam.

to the fullest extent, but all tensional stress is thrown upon the steel. The steel being embedded in the concrete will not rust, and therefore the permanence of the structure is secured.

For rock bottom the apron may be omitted (Figs. 218 and 219).

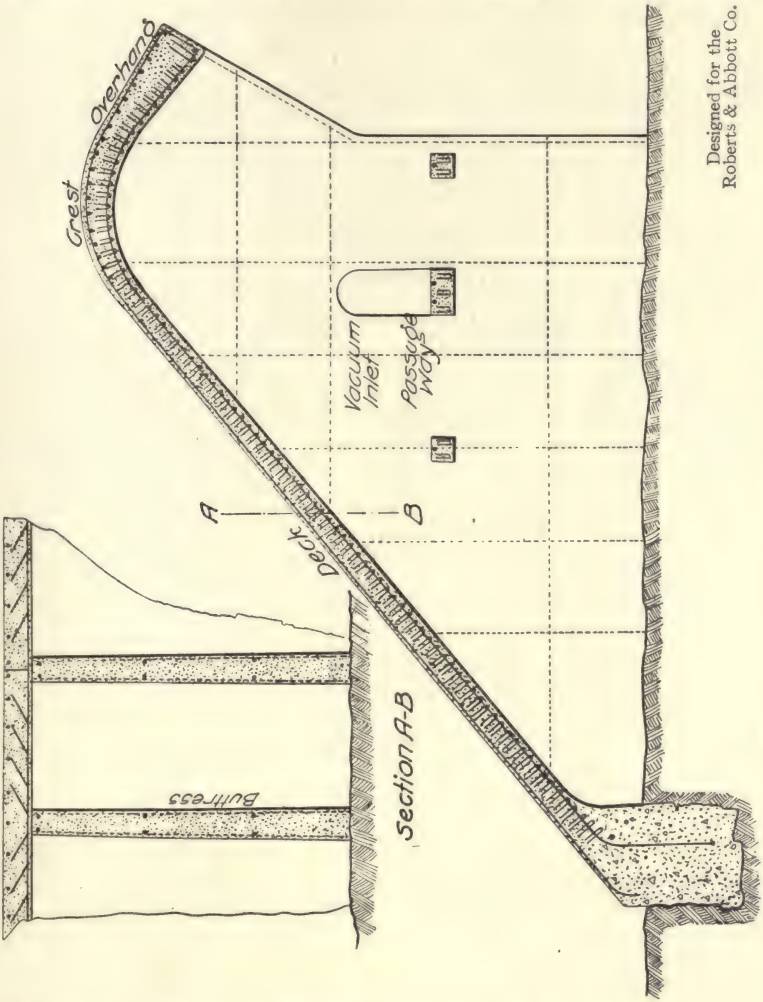
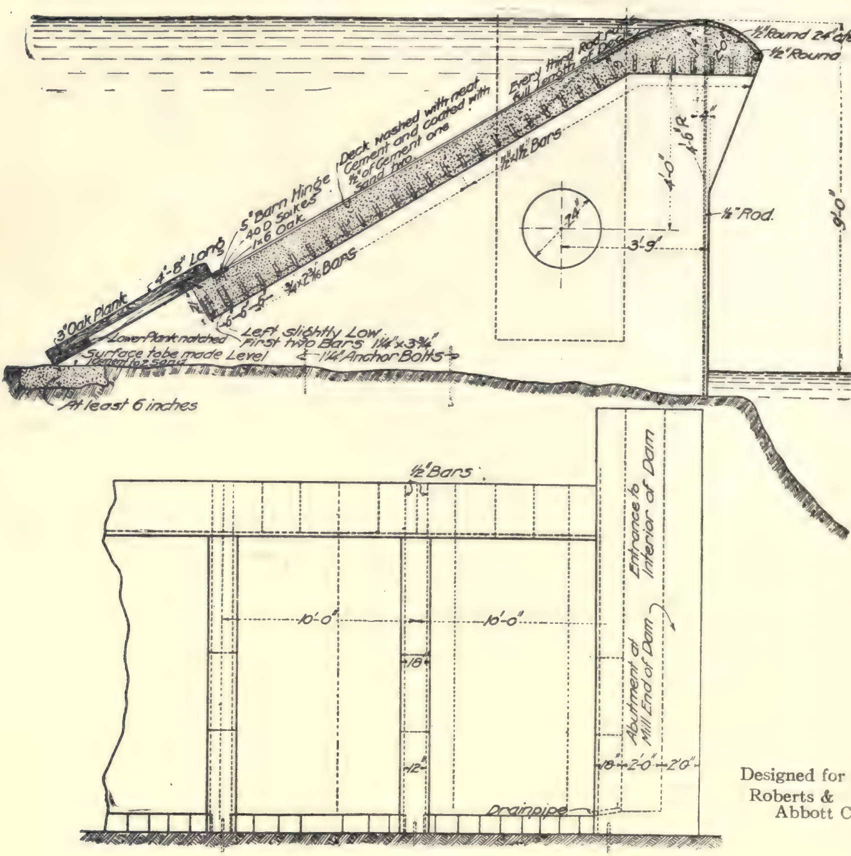


Fig. 218.—Reinforced gravity dam 48 feet high.

The passageway is for the free admission of air to the interior and to permit inspection.

There are certain conditions under which even a concrete-steel dam will fail to give perfect satisfaction. Thus, if the

bottom yields ever so little, the dam will crack. The steel will hold it together, but leaks will start, which, in time, will cause damage and loss of power. Again, such a dam must necessarily be constructed entirely in place in the river bed, which means more or less risk from floods during the building.



Designed for the Roberts & Abbott Co.

FIG. 219.—Reinforced gravity dam.

The design (Fig. 220) shows a dam that is composed of individual reinforced members, each of which is built on shore, and thoroughly tested and seasoned before placing in the dam. The deck is made up of these segments C, and the apron deck is made of similar segments, the only difference being that the apron

segments are of less depth and have more anchor bolts. Each segment has recesses, *F*, moulded along the sides, so that when cemented between, as *G*, the whole is bound together. Each segment is a beam eight inches deep and 12 inches wide, in the

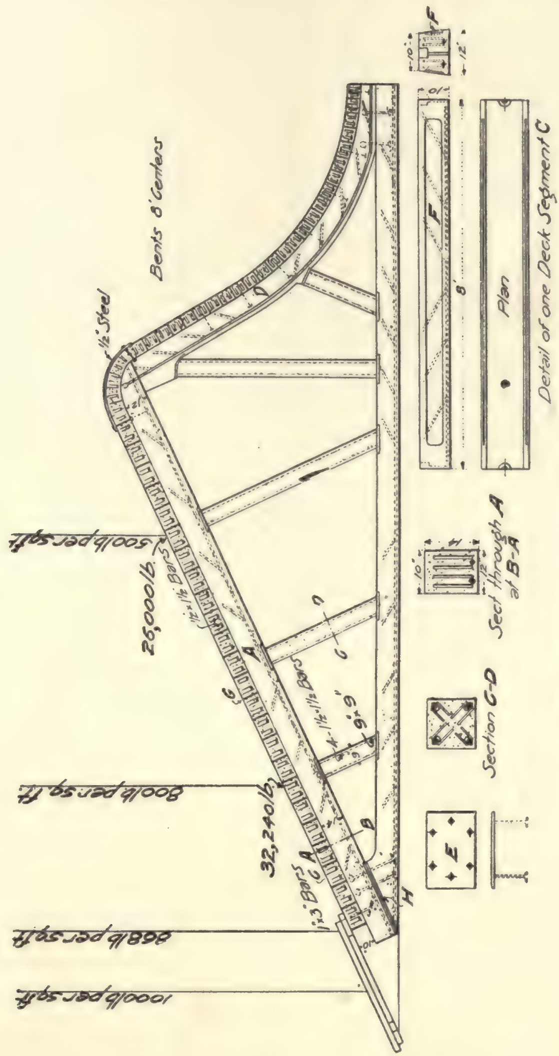


FIG. 220.—Segmental reinforced gravity dam.

dam shown, and as long as the distance between bents, each sustains eight times the actual pressure per square foot, and may be designed from tables in Chapter IV.

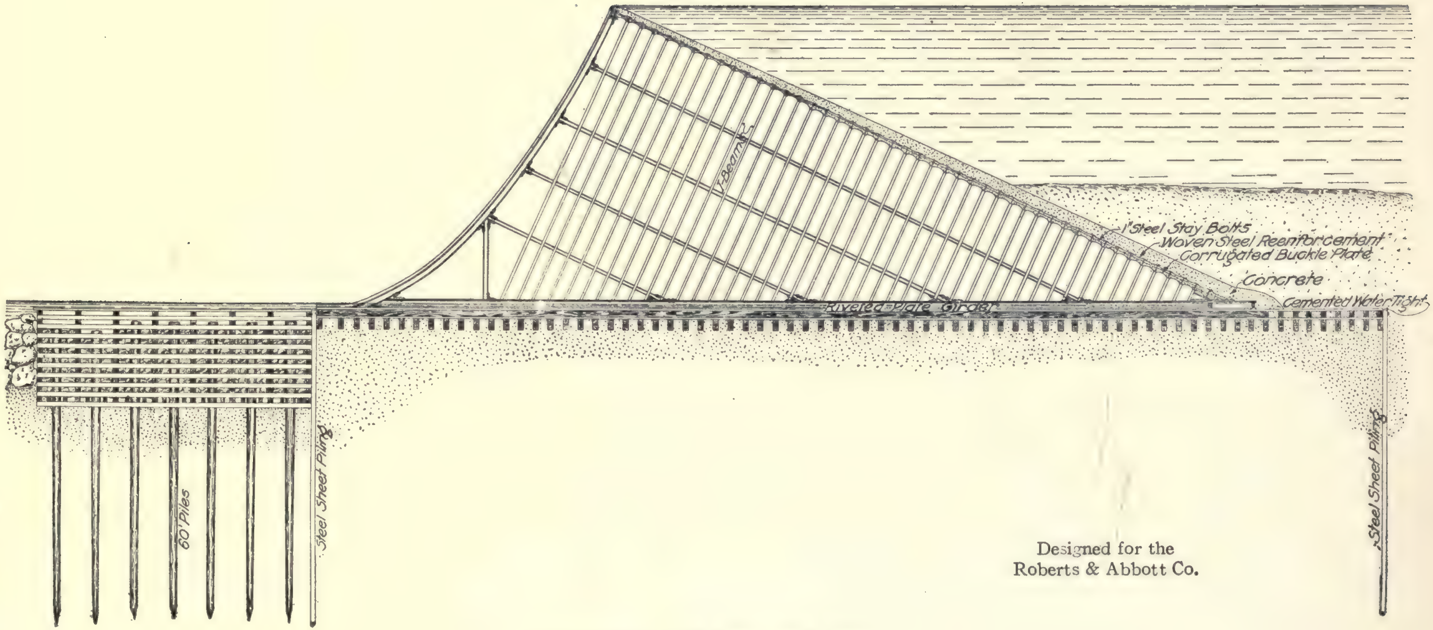


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FIG. 221.—100 foot steel dam.

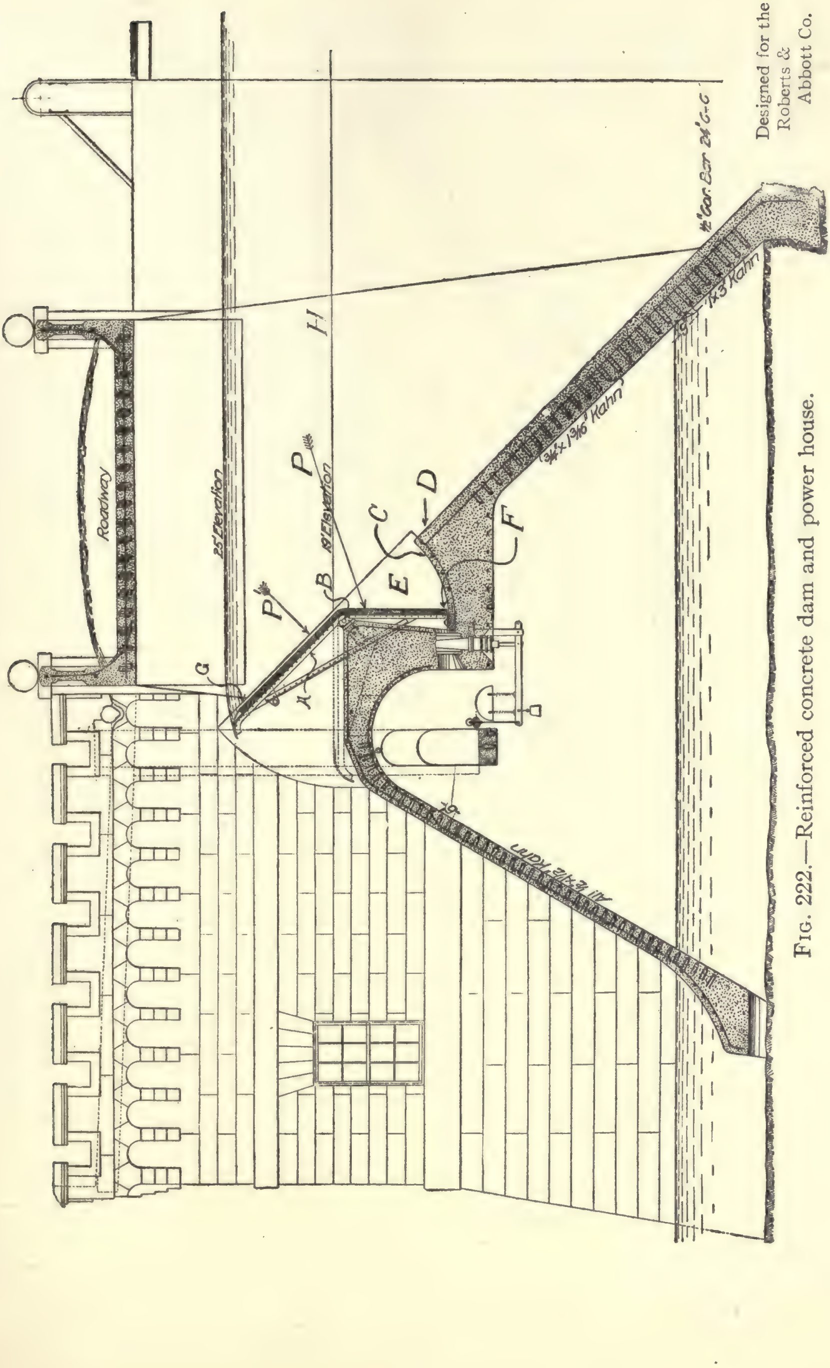

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FIG. 222.—Reinforced concrete dam and power house.

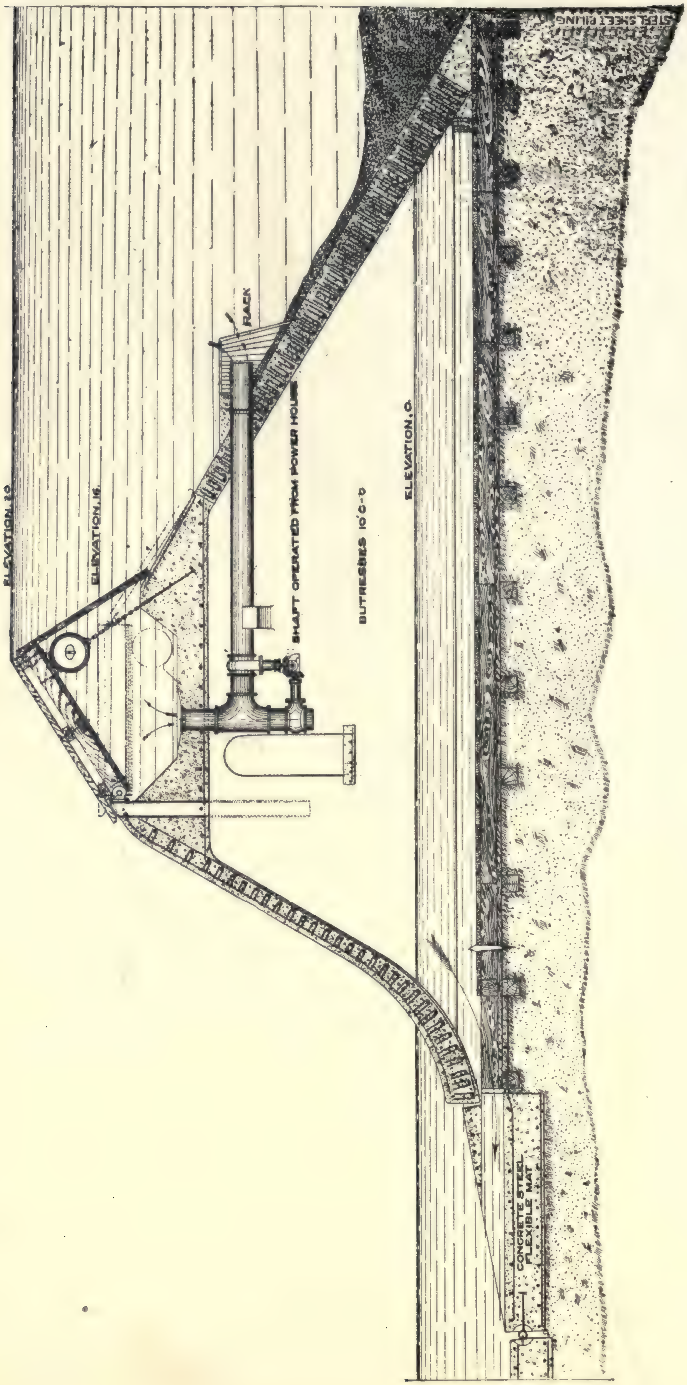

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FIG. 223.—Reinforced gravity dam with movable crest.

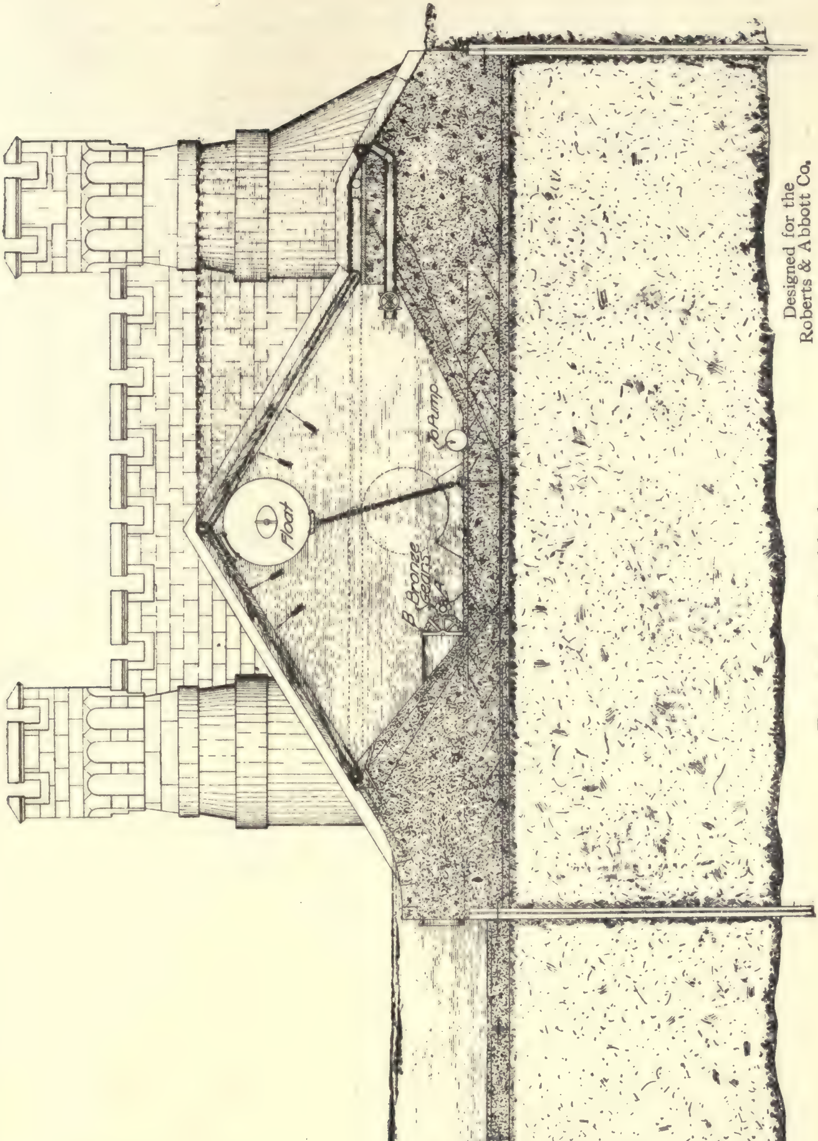

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FIG. 224.—Movable dam.

Referring to Table XXIXa, page 121, we find that the segments near the toe of the dam should be 10 inches deep and be reinforced with 1.31 square inches of mild steel to sustain the load. The first segment supports three feet of the the foot boards, or $3 \times 8 = 24$ square feet of surface; therefore, $24 \times 1000 = 24{,}000$ pounds is the pressure upon it. For this beam of 14-inch depth two inches of steel are required. At each end of the segments is molded a half recess to permit of bolting to the deck and apron plate. When the segments are all in place on the dam a rich cement-sand mortar is filled in between, making the joints water tight. Now as these segments only run from one bent to the next, it is evident that one bent could sink a good deal without impairing the strength of the dam. Also if the deck warps, the only place a crack would occur would be between the segments and not across them, where the reinforcing is.

In a dam of the size shown (Fig. 220), the sill *A* is made all in one piece, but for higher dams it may be made in more pieces. Where each post comes, a steel plate *E* is molded. The four holes have the same spacing as the reinforcing rods in the posts, so that when the post is set up the rods slip onto them. At *H* a plate is molded into *A*, so that the bolts connecting it with the sill will have greater shearing value. The holes for these bolts are cored into the plate. The apron plate is made in the same way, a number of anchor bolts being molded in, to hold the apron segments. The sill is 10x10-inch and reinforced with 1x3-inch bars. The dimensions of the posts are obtained from the formula: Safe Load $= 350$ (area of concrete $+ 15 \times$ area of reinforcing steel).

This dam may be placed upon a timber mat the same as a timber dam. Gravel should be placed on the deck so that if a crack should occur leakage will be prevented.

WING DAMS.

On navigable rivers, wing dams are sometimes built to avoid obstructing navigation. These dams are run part way across the river and frequently quite a distance up the stream. The head thus acquired is necessarily low, but usually in such cases there is plenty of water. At Rock Island, Ill., there is a very large power created by a wing dam and used by the United States Government at its arsenal.

The wing dam is built in the same way as others described except at the end which receives the full force of the current. At this point every precaution must be taken to provide against undermining. The pier A, Fig. 225, is built first, a coffer being built so that the bottom of the river may be excavated.

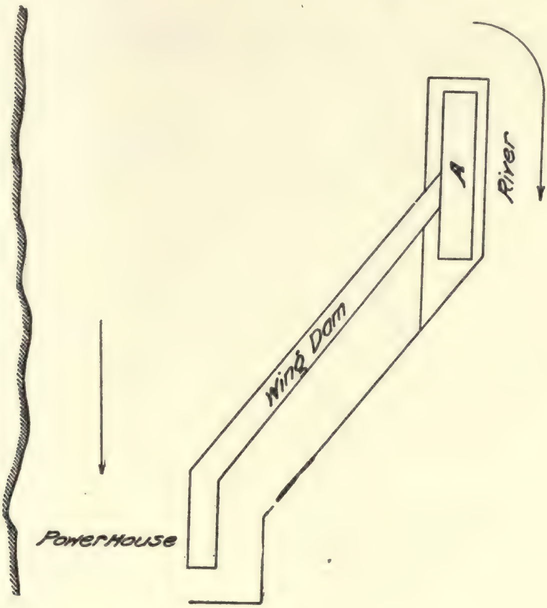


Fig. 225.

The foundation of this pier must go down below the level of possible wash. Having built a safe end the building of the dam possesses no unusual difficulties.

BOW DAMS.

Dams, especially masonry dams, are often bowed up stream, as shown in Fig. 226, in an exaggerated form. The idea is to get the strength of an arch. When the ends of vertical faced masonry dams are given a secure anchorage, as in Fig. 226, there is no doubt but that a great increase in strength is secured by the arch, but the ends must make an angle with the stream such that a line $C\,D$, coinciding with them, passes inside the center of curvature. The water pressure, being perpendicular to the surface at all points, presses every part towards the center

of curvature, and if the ends are given a less slant, as $A\ B$, the pressure at P tends to shove the end away from the cliffs and the dam is no stronger than if built straight. The author knows of two bowed masonry dams, each of which failed at both ends.

For timber, or gravity dams of any material, the bow adds no degree of safety. The old-style crib dam, unless very short, would gain little by the arching, as it would fail, due to local weakness at some one point, and disintegrate without giving an

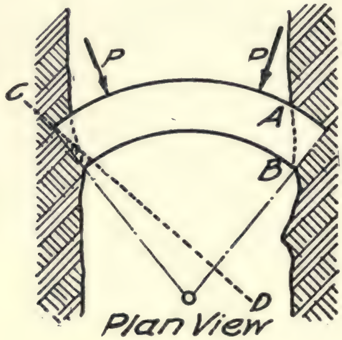


FIG. 226.

end thrust. The gravity dam depends on the vertical water pressure to hold it in place, therefore the arch would add nothing to its security.

MASONRY DAMS.

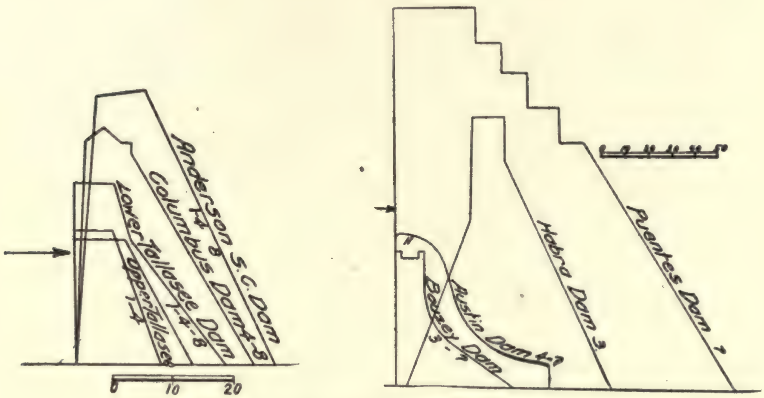
The search for permanence has developed the masonry dam. Its great first cost would have made its use impossible had there not been a strong prejudice against all other forms. The feeling of security given by the use of masonry often made a proper disposition of the materials a secondary consideration, with the result that the list of masonry dam casualties contain almost as many failures as that of timber dams.

With the passing of the forest and the decreasing cost of concrete, however, it becomes more and more important that we perfect the masonry or concrete dam.

Assuming that the concrete or masonry is properly laid, there are eight prime factors which must be determined and provided for before the actual work of construction begins:

1. Wall being sheared by the horizontal push of the water.
2. Undermining below the dam, due to weak apron.
3. Resistance to sliding on its base.
4. Effect of vacuums.
5. Effect of flotation on the weight of materials.
6. Effect of ice expansion.
7. Liability of seepage under the dam.
8. Weakness of green concrete or masonry.

To convince the reader that it is worth while to study the above points well before indulging in hasty construction, the cross-sections of a few masonry or concrete dams which have failed, are given in Figs. 227 and 228, these costing millions of



FIGS. 227, 228.

dollars and a great many lives. The cause of these failures may be found among the above eight factors, and the probable factors which caused the failure have been indicated on each section.

The author contends that the cause of so many disasters is because the factors 4, 5, 6, 7 and 8 have not been understood, and by merely building to oppose the hydraulic pressure, instead of turning them into factors of safety.

By referring to these sections it will be seen that without an exception the up-stream face of the dams are vertical, or practically so, and that all are apparently of heavy proportions, when the water pressures alone are considered. In discussing the above eight factors it must be borne in mind that there are

but two forces, the amount of which can be figured with any degree of accuracy. These are the hydraulic pressure and the crushing strength of the masonry. It is an undisputed fact that the tensile strength of masonry or concrete is so uncertain that it can not be relied upon at all. Also the shearing strength is unreliable. The weight of the material is a quantity which is equally difficult to compute, owing to the factor of flotation. Of course, it is at once apparent that the portion of the dam which is below the surface of tail water is lifted up by the amount of the weight of the displaced water, and that this lifting effect is increased by the backwater caused by floods.

It has been contended that the water which soaked into the masonry owing to the hydraulic pressure did not affect the weight of the material thus soaked, but lately leading engineers are taking the stand that the weight is very materially affected, though as to just what extent they are still at variance. This loss in the effective weight of the material the author calls the factor of flotation, as the tendency is to float the masonry, and has demonstrated to his own satisfaction that it is not safe to figure the effective weight of the affected masonry at more than two-thirds its actual weight.

In the following table is given the amounts of water which a cubic foot of sand and some common rocks will absorb:

TABLE XXXVII.

Material.	Water absorbed per cubic foot.	Material.	Water absorbed per cubic foot.
	Quarts.		*Quarts.*
Sand	10	Dolomite	1 to 10
Potsdam sandstone	2 to 6	Chalk	8
Triassic sandstone	4	Granite	1/100 to ¼
Trenton limestone	¼ to 1¼		

Bearing in mind these points, it will be seen how very uncertain is the material which has been looked upon in the past as the most trustworthy agent to resist the hydraulic forces. Its one factor upon which reliance may be placed (the crushing strength) has never been made use of in the past, as all masonry dams which have failed, however scant their dimensions may have

been, were absolutely safe against crushing. This is because the structures were built so weak in other ways that the limit of the crushing strength could not possibly be reached before there was a wash-out due to some other cause.

We will now take up the eight factors in their order and consider their importance in the design of dams:

1. If the dam should give way at G D, Fig. 232, owing to the horizontal down-stream push of the hydraulic pressure, it is sheared at that point. Even an approximate value for the shearing strength is impossible to predetermine, as it depends on variables, such as evenness of mixing the mortar or concrete; parting lines formed between bodies of the materials laid at different times; strains set up in the materials, owing to uneven setting, etc.

2. One of the most common causes of disaster is due to the dam being undermined on the down-stream edge of the structure, and to the sucking force of vacuums. To prevent this a massive extension mat must be provided. (See design on page 234).

3. Referring to the sections of the above dams it will be seen that *all* the pressure is directly down stream, because it is a well known law that *water pressure is always perpendicular to the exposed surface.* Therefore the only thing to hold these dams in place is the friction between the bed of the stream and the dam. The Austin dam is a very noteworthy example of a dam which failed by sliding on its base. A large section slid bodily down stream and still stands erect, several hundred feet below its original position. Of course, the heavier the materials, the greater the friction. The crushing strength of the masonry, however, does not enter into consideration.

4. In considering the fourth factor the reader is referred to the result of the Cornell experiments, given on page 14. These experiments prove that, for even a small fall and overpour, a powerful vacuum is formed. In the case of the eight-foot dam it had to sustain the equivalent of five feet of head in addition to the eight feet, or over half again as much. This means 312 pounds per square foot of surface. Where the dam is from 40 to 100 feet high and the sheet of water flowing over the apron is six or eight feet thick, the vacuum must be almost perfect. It has been found that the vacuum adds fully 1000 pounds per square foot to the pressure on the dam where the dam is 20 or 30

feet high and the sheet four feet or more thick. The presence of a vacuum can be qualitatively demonstrated by noting the inrush of air at the inlets provided in dams for the relief of the vacuum. The Columbus, Anderson and the Upper and Lower Tallassee dams all failed within a day or two of each other, and in each case the hydraulic pressure (owing to the backwater) was less than at any other time in the history of the dam. Then why did they fail? Simply because of the terrible vacuum pressure and the floating effect produced by the increased backwater below the dam.

In order to more clearly demonstrate the action of vacuums, the following reasoning is given: Referring to Fig. 229, the section of the overflowing water is similar to that of the dam.

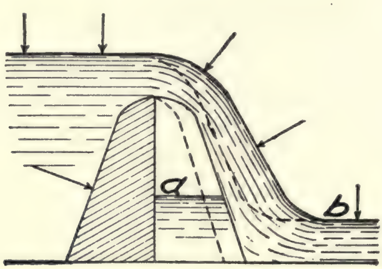


FIG. 229.

Now there are two fundamental laws of motion which we must accept as being correct.

1. A body under motion will continue so unless acted on by some external force.

2. Action and reaction are equal and oppositely directed. The action of the air pressure when a vacuum is formed is to deflect the mass of falling water from its normal path, as shown by the dotted lines. Of course there can be no question but that to deflect tons of rapidly falling water force is required. This force shows itself by the difference in elevation of the water behind the pour, and that below, as the levels *a* and *b*. According to the second law there must be some reaction to the air pressure on the overflow. Suppose Fig. 229 to take more simple form, as in Fig. 230, the imponded water not being considered.

Now the water between the overflow and the dam is held to the level *a*, by the partial vacuum *V*. In other words, the water is sucked up in between the dam and the overpour, creating a suction on, not only the wall of water on the right, but also on the wall of masonry on the left. Can there be any question on this point? Mr. Frizell, in his very excellent treatise on hydraulics, says, "Its greatest possible deleterious effect would be to press the stream against the down stream face of the dam," and claims that the vacuum tends to sustain the dam. Of course this view is faulty, as it does not consider the reaction on the pond-side of the dam. Filling the pond above the dam does not change the above reasoning, as the air pressure is perfectly transmitted through the imponded water.

The water and the wall tend to be sucked into the vacuum, and to resist this suction a certain factor of safety must be allowed.

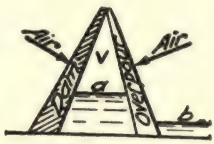


Fig. 230.

"Suction is the act of exhausting air from a cavity, but it acts upon the air within the cavity, not upon the walls of the cavity, nor upon any substance heavier than the air; a piece of paper upon the floor of the cavity would not be disturbed by the suction. Suction is the primary cause and vacuum is the effect. The breaking of the walls of the cavity is the effect of a secondary cause—atmospheric pressure—forcing in the walls to fill the vacuum.

Its action may be better understood from the following illustration: A piece of pliable leather having a cord attached to its center, when saturated with water and pressed upon a stone, adheres with such force that in many instaces it is possible to lift the stone. The pull on the cord reduces the atmospheric pressure on top of the stone, and in cases where the stone is not heavier than the total pressure on the leather it is possible to reduce the pressure on top of the stone to such an extent that the atmospheric pressure from below will lift the stone.

The operation of this familiar experiment is precisely the same as that of vacuum suction on dams. The sheet of water is represented by the piece of pliable leather, the dam by the stone against which the sheet is pressed, and the projectile force of the sheet by the pull upon the cord.

If, in the experiment, the atmospheric pressure is greater than the weight of the stone, the stone will be lifted and no vacuum formed, and so the dam may receive a pull equal to the projectile force of the sheet, even though the over-pour does not leap away from the apron and, without causing a vacuum, develop an unseen, unsuspected force, which may have the power to destroy the dam.

Since it is evident that suction brings into action a secondary force against the outside face of the cavity or wall only, we are driven to the conclusion that the facings on the lower half of aprons are not displaced by suction.

The more probable cause of the defacement of aprons is found in a deficient resisting power of the masonry, the erosion of weak mortar from poorly-constructed joints, and the development of vibrations caused by great columns of water pounding upon the facings with a force of many thousands of tons. This theory is the more probable since the facings are not torn off at points above the lower half of the apron where vacuums do occur, but are torn off at a point where, in all probability, they do not occur."*

Now as to the amount of this air pressure: At first thought it would seem that the effective pressure would only be figured as pressing the dam from the crest down to the surface of the water *a*, Fig. 231, but this is not so. Referring to Fig. 231, the column of water behind the over-pour caused by the vacuum reacts on the dam, tending to neutralize the vacuum pressure. The pressure against the dam considering only the vacuum pressure and that due to the level $E\,D$, at the water level $C\,D$ the pressure is zero, but at level $A\;B$ and $H\;I$ it equals the pressure due to the level $E\,D$ and therefore equals the vacuum pressure. The line $E\,A\,H$ therefore is the line of pressures at any point along $E\,F$, due to the backwater against the dam, and the line $C\,F$ is the line of vacuum pressure. As the

*E. R. Beardsley.

air pressure acts against the up-stream face of the dam, through the emponded water, it must be perpendicular to that surface of the dam, and will be uniformly distributed along the face. We may, therefore, represent this vacuum by the line $C' G'$ parallel to this face and at a distance from it equal to the vacuum pressure. The line $C' F'$ is $C F$ transferred. Now the center of gravity of this shaded area $F' E' G' C'$, which represents the entire vacuum pressure, may be found and the whole pressure considered as acting at that point, and tending to slide or overturn the dam.

The amount of vacuum pressure for any particular dam is very difficult to determine, owing to the lack of data on the subject. The author has made the following experiments with dams fitted with air inlets:

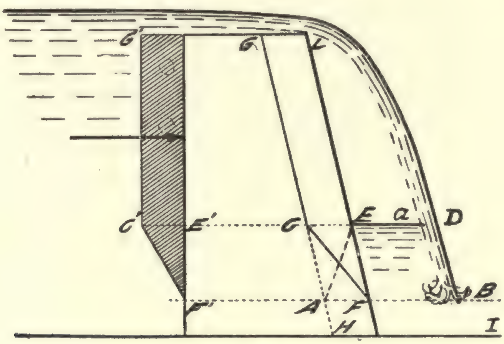


FIG. 231.

A dam 565 feet long, with a total head six feet, and a head over the crest of three feet, showed a vacuum pressure of three feet of water or 1.31 pounds per square inch; a dam 300 feet long, with a total head of 12 feet, and a head over the crest of 30 inches, showed a vacuum of three feet of water or 1.31 pounds per square inch; a dam 275 feet, with a total head of 30 feet, and a head over the crest of four feet, showed a vacuum of 14 feet of water or six pounds per square inch. At Cornell a weir six feet high, with 18 inches of water, showed a vacuum pressure of two feet of water or 0.86 pounds per square inch; and a dam eight feet high, with two feet of water, showed a vacuum of five feet of water or 2.16 pounds per square inch. From these examples a fair guess may be made as to the probable pressure due to vacuums.

One of the results of the formation of the vacuum is to set up vibrations which may seriously affect the stability of the structure. Water falling perpendicularly into the river bed or upon the apron, gives a series of rapid blows, keeping the entire structure in a tremulous condition. A body placed on the floor may be easily moved if the floor is vibrated. Now add to the effect of these constant vibrations, the heavy rhythmic vibration caused by the vacuum, and the conditions are perfect for the down-stream movement of the dam. When the overpour is not too heavy the vacuum forms more or less perfectly until the sheet of water can resist the inward pressure of the air no longer, when it breaks through the sheet, thus restoring the equilibrium. This process is repeated with great regularity and results in an intense horizontal push and pull on the dam. If the vacuum

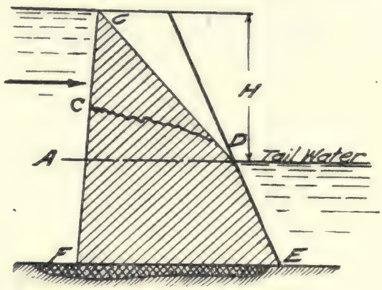


FIG. 232.

inlets are too small, this action takes place, causing a puffing noise, similar to a locomotive pulling a heavy train up hill.

5. It will at once be apparent that that portion of the dam which is entirely below tail water (see Fig. 232) is floated up with a force equal to the weight of the displaced water, but it will possibly require a little study before the reader will understand how the water which seeps into the solid masonry can cause a diminution of weight in the affected portions. One of the most widely known and popular experiments to illustrate the action of water pressure is the bursting of a stout keg by means of a high but thread-like column of water. This is the action which takes place through the finest seam of the most minute interstices and exerts a lifting or floating effect on the particles which go to make up the mass of the structure. Re-

ferring to the sketch (Fig. 232) all that masonry on the up-stream side of the line *D C* will be affected in this way, and there is such a small amount of dam left unaffected that the only safe way is to suppose the entire structure as affected. A fairly safe method is to figure the mass of the dam above tail water as losing one-third its weight, and that below tail water as losing 62 pounds per cubic foot.

Mr. J. B. Francis held that solid concrete deposited on bed rock would be lifted or floated, and to prove this, placed a pipe provided with a pressure gauge, in the concrete of a dam and found that the gauge registered the full pressure.

6. Ice expansion, the sixth factor, becomes, in our northern climate, a most deadly foe to all dams, and especially to masonry dams. The co-efficient of ice expansion is nine-tenths of an inch to the 100 feet, and a sheet of ice one mile long will expand nearly four feet. It is evident that dams do not receive all this expansion. If they did the first severe winter would destroy them. A large per cent. is already expanded when the sheet is first attached to the dam, but the expansion will be continued as long as severe weather lasts.

The following is taken from the *Architect and Building News:*

' A short railway was once built in the Province of Ontario which crossed a fresh-water pond, known as Rice Lake, by a bridge two and one-half miles long. The bridge was mostly com-composed of trestle-work, very strongly built, with uprights driven in hard bottom and thoroughly braced. The middle portion, over the deepest part of the lake, was composed of trusses eighty feet in span, supported by piers measuring 12x24 feet and filled with stone. Early in the first winter after the bridge was built the lake froze over to a depth of about seven inches. Before snow came to protect the ice, the weather moderated, the sun shone out brightly, the ice expanded, and in a few minutes the bridge was in ruins its whole length, the trestles being pushed over in the direction of the principal expansion.

Afterwards the trestles were repaired and filled in with gravel, the top of which is eight feet above the level of the water, yet the expansion of ice during sunny days is so great that it frequently creeps up the embankment, and, by successive movements, is pushed upon the rails."

The extent of the lake is not given, but with an open ice field of from three to four miles, and all the conditions favorable, the expansion would be as stated. Many instances have been known on lakes and rivers, where, under the enormous pressure, acres of ice have been hurled into the air as if by an explosion of dynamite. An instance of this kind came under the writer's observation a few years since, on the Kankakee River, Illinois, at the mouth of Baker Creek.

The expansive power of ice is plainly shown along the shores of the small northern lakes, more especially those having firm bluffs on one side and low, marshy lands on the opposite side, which receive the full force of the expansion, and which after many years of repeated action have pushed inland the frozen earth of the shore up into parallel windrows, dykes or moraines from one to several feet in height.

A singular phenomenon attending expansion is that it is greatest when the sun comes out and shines upon the ice while freezing. Short dams suffer less than long ones, because the abutments and banks offer great resistance, and expansion is thrown in the direction of least resistance. If the dam is long, the shore protection is diminished and its central portions more exposed. For this reason the dam is rarely moved at the abutments, but always at the center, with decreasing ratio towards the ends.

If the dam is on rock bottom and bolted down, however fast, the repeated strain, winter after winter, tear it loose from its fastenings. If built on pilings, the movement is imparted to them, loosening the foundations, disturbing the general solidity of the structure, causing leakage and assisting in the work of general dissolution.

The only way the vertical faced concrete dam can be built to be safe against ice expansion, is to give it enough strength to actually crush the ice against its up-stream face. Otherwise the ice will crush the dam, as there is no give and take to masonry. Trautwine gives 12 to 14 tons per square foot as the compressive strength of ice. This means that if a field of 12-inch ice gets a grip on to the crest of the dam it can push it with a force of from 12 to 14 tons for each foot of the length of the dam.

7. Seepage exists to a certain extent under all dams, and while it does not always indicate a weakness may prove to be

a destructive factor. If the resistance to the escape of the water below the dam at E Fig. 232, is greater than that to its entrance under the dam at F, there will be a back pressure created in direct proportion to this difference. If in Fig. 232 we suppose the water has free access under the dam as shown by the double shaded portion, then the whole bottom area of the dam will be lifted up with a pressure due to the *head* of water H and the dam floated out of position. Therefore, the up-stream toe of the dam must be made more nearly water-tight than the down-stream edge. In fact, drains should be placed under the dam to conduct away the water which does seep through the toe at F. Fig. 237, shows a dam of excellent design with the cutoff duct a to catch the seepage and the seepage trench b to make the toe as tight as possible. The seep holes c need be only about six or eight inches in diameter and about 10 feet between centers lengthwise of the dam. With this design the effect of vacuum is allowed for and the dam allowed to rest with all possible weight upon the foundation.

8. The eighth factor we believe to be a very important one. The time taken to complete a heavy masonry dam varies from two to five years. To hasten the construction, large quantities of masonry or concrete are laid each season, with the result that the interior of the dam does not get a chance to dry out, and hence has very little strength. However, it is rushed to completion late in the season and is compelled to withstand whatever floods may come its way.

If there are no floods, well and good; but if, as in the case of the Cloumbus, Anderson, Upper Tallassee and Lower Tallassee dams, the floods come at once, the dams are in great danger of destruction. The four above-named dams were new dams, but all were destroyed by the same flood.

In Fig. 233 we have attempted to show graphically the proportions of the three great agents of destruction—water pressure, ice expansion and vacuum. Of course, the dam does not have to be of such great proportions, as, fortunately, when the ice can fasten to a dam there would be no vacuums, there being so much water that the ice could not fasten to the dam; but the sketch will show the importance of each of these factors. Each section is completed without a factor of safety and shows a dam just ready to be overturned by the particular force. In calculating

the ice expansion, the very low value of 8000 pounds pressure per foot of dam has been taken. Trautwine gives 24,000. The vacuum pressure has been taken as 625 pounds per square foot of surface exposed and the exposed surface called the area 10 feet below crest of dam.

However well designed a vertical faced masonry or concrete dam may be, yet we must acknowledge that it is in the nature of a dangerous experiment. Every great masonry dam that was ever built has called forth heated discussions among the most noted engineers. The Croton dam was delayed years on account of such conflicting opinions. As this book goes to press the professors of University College, of London, England, come out with the startling announcement that all previous dams have been designed without taking into account a force tending to burst the dam vertically. Following is given a clipping from

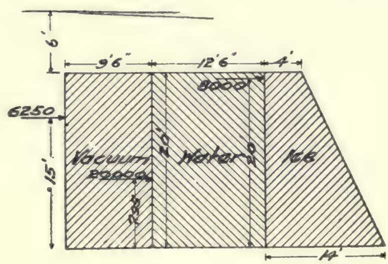


FIG. 233.

a London paper, which shows how the best laid plans go oft agley.:

" The new theory regarding the strain upon masonry of dams, brought forward by Atcherley and Pearson, mathematicians of the University College of London, has, at least for the time being, put an end to Sir William Garstin's plan of raising the gigantic dam at Assouan, which has already proved such a blessing to Egypt.

" This was an important part of a huge plan for further irrigation of Egypt, destined to bring millions of acres under cultivation. Lord Cromer's last report dealt minutely with the scheme, the outlines of which then were cabled.

" Sir William, through calculations of the engineering staff, was satisfied that according to all accepted theories of dam

construction the factor of safety was amply sufficient to permit the dam being raised, but in October he was informed of the new theory that the vertical sections of dams under water pressure were more severely strained than the horizontal parts.

" Therefore, while the dam was designed according to rules hitherto applied and may be safe as regards cracking horizontally, it may crack vertically.

" The Egyptian Government asked Sir Benjamin Baker to give an opinion on the raising of the dam. After inspecting the dam Baker reported that all thoughts of raising it should be postponed another 20 years.

' He is of the opinion, basing it on the new theory, that there now is little hope of raising the dam to any appreciable extent, although calculations submitted to and passed by him before the new theory was correct in all respects. He adds that the vibrations on the masonry dam are due to the rushing water in the sluices, and that the dam as constructed is perfectly safe.

" 'There should be ' he said, ' perfect confidence and no need of anxiety in the permanent stability for centuries without difficult or costly works for its maintenance.' "

Masonry Dam Design in Detail.

The design of masonry dams will now be treated in detail.

First, take the case of a dam (Fig. 234) having the water level

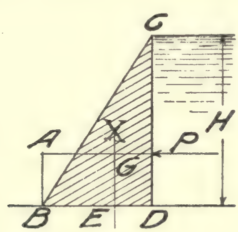


Fig. 234.

with the crest and the up-stream side (face) perpendicular. The pressure at $P = \frac{H}{2} \times 62.5 \times H$ and, in effect, is applied at the point $\frac{1}{3} H$ from the bottom. The pressure P acts perpendicular to the face, and in turning the dam over, uses the lever $A\ B$. That is, the pressure P in pounds times $A\ B$ in feet, is the overturning moment.

The center of gravity G is found by drawing lines from C to the middle of $B\ D$, and taking $\frac{1}{3}$ of it. The weight of the masonry (due allowance being made for the flotation factor) may now be supposed to act at this point, and in holding the dam against the overturning force of the water, use the lever $B\,E$, $E\,X$ being drawn through the center of gravity and perpendicular to $D\,B$, therefore the resisting moment due to the weight of the dam is $W \times B\,E$ where W is the *effective* weight of the dam. The factor of safety of the dam is

$$\frac{W \times B\,E}{P \times A\,B}$$

In all these examples one linear foot of the dam is considered, *i.e.*, the pressure for 100 feet of dam would be $100P$.

Suppose the dam to be turned around, as in Fig. 235. The water pressure acts perpendicular to the face and at a point

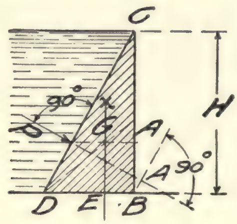


FIG. 235.

P, $\frac{1}{3}$ the height as before. P acts in overturning the dam with the leverage $A\,B$, which is the perpendicular distance between the projection of P and the toe of the dam B. It will be seen that the flatter the face $C\,D$ the greater will be the moment of the vertical force. That is, the more nearly the dam approaches the gravity type the less the tendency of the water to tip it over.

The center of gravity is found as before and the perpenidcular, $x\,E$, is drawn. Then the effective weight of the dam will act, in resisting the water pressure, through the leverage $E\,B$, therefore the factor of safety is

$$\frac{W \times E\,B}{P \times B\,A}$$

Now take the case of Fig. 236, where the water is flowing over the dam at a depth h.

In this case P is not applied at a distance of $\frac{1}{3}H$ from the base, but at a distance

$$Y = \frac{H}{3}\left(1 + \frac{h}{H + 2h}\right)$$

The overturning moment of the water then $= P \times A\,B$.

To get the center of gravity of a section like this, draw a line connecting the centers of the lines $D\,B$ and $F\,C$. Lay off $R\,F$ equal to $D\,B$, and $D\,S$ equal to $F\,C$. Then where $R\,S$ crosses the center line, will be the center of gravity. The resisting moment of the dam $= W \times E\,B$ and the factor of safety is

$$s = \frac{W \times E\,B}{P \times A\,B}$$

If this section is turned as in Fig. 235 the same reasoning is true.

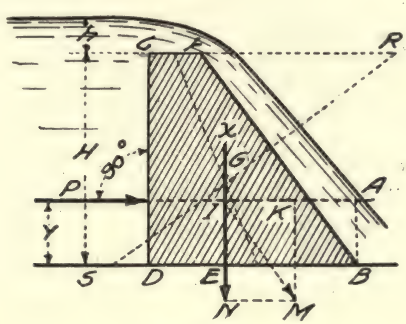


FIG. 236.

A common method of determining the safety of any section is to lay off to scale from I (Fig. 236) the line $I\,K$ equal to the pressure P, and from I the line $I\,N$, to the same scale representing the effective weight of the dam. In gravity type the weight of the water should be considered. Then the resultant, $I\,M$, should intersect the base $B\,D$ in the middle third. Such a method is faulty, as it does not take into consideration the factors 2, 3, 4, 6, 7 or 8 (page 239), but will serve for the basis of further design. After getting this preliminary section the forces due to vacuums, ice expansion, character of bottom, etc., can be considered and extra area of section added.

As shown by Fig. 235 the dam is safest against the water

pressure with the slope up stream. Especially is the factor of safety against sliding increased, but there is the overpour to be considered. In order to conduct the water away from the dam in safety, an apron is required, and if, in addition to the apron, the dam is sloped up stream the result is a dam too expensive to build. For this reason practically all masonry dams are built with the face about perpendicular, the slight up-stream slope given to some being merely a temporizing between cost and safety.

Therefore to design a masonry dam, first design the apron, giving the necessary slope to prevent vacuum (Fig. 237), and then lay off the section *c f d b*, as found for Fig. 236, having the resultant of the water pressure and gravity of the dam strike

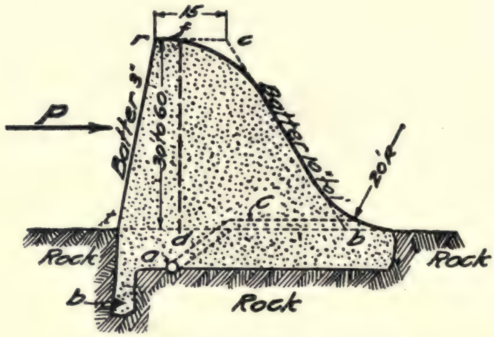


Fig. 237.

in the middle third of the base. Now add the section *f r t d* of area sufficient to make the dam safe against ice expansion, seepage, shearing and the weakness of green masonry. The flotation will have been allowed for in determining *W*, and undermining will have been provided for in building a liberal apron. Such a masonry dam should be safe, providing foundations are good, and the materials are well laid. After the section has been decided upon it is well to investigate the critical points to see if the safe crushing strength has not been exceeded.

In General Gillmore's reports we find the following table of the crushing strength in tons per square foot (2240 pounds) for various stones:

99 specimens of granite.		Highest 1541 tons.	Lowest 497 tons.
43 "	" limestone.	" 1600 "	" 221 "
12 "	" marble.	" 1284 "	" 488 "
62 "	" sandstone.	" 1136 "	" 251 "

The lowest values are from the poorest quarries in the United states, so it would seem that by taking one-tenth of the minimum values one would have reached the extreme of safety. However, in the design of the Quaker Bridge dam (Croton dam) for the New York City water supply, the engineers limited the safe crushing strength to 13.5 tons or 30,000 pounds per square foot.

Crushing stresses are determined as follows:

In Fig. 238, let L equal the base of the dam, d the distance

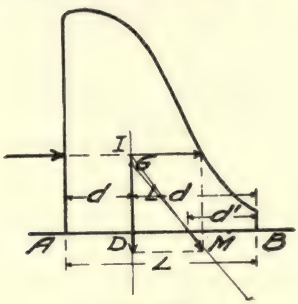


FIG. 238.

between a point where a perpendicular passing through the center of gravity cuts the base and the up-stream edge of the dam. Then $L-d$ is the distance from B to D. Let p be the maximum unit stress then

$$p = \frac{W\,(L-d)}{L\,d}$$

and

$$p' = \frac{W\,(L-d')}{L\,d'}$$

wherein W is the effective weight of the dam.

The resultant $M\,I$ is found as in Fig. 238. By measurement on the drawing the distance d is found, and substituting in the above, p for the segment d may be determined, this gives the maximum compressive stress in the base at that point in pounds per square foot, when the pond is empty. To find the stress near B due to the water pressure measure d' and substitute in the above formula. The answer will be the pressure near B in pounds per square foot.

In designing a very high dam the first operation is similar to the preceding: the top section assuming $H = 100$ feet (Fig. 239) is taken first. Find the point of application of the water pressure P, as in Fig. 236. Find the center of gravity g and g' of each of the triangles $A\ B\ C$ and $A\ E\ D$ as already explained. Find the center of gravity of these two triangles as follows:

$$\text{distance } g\,G = g\,g'\left(\frac{W'}{W+W'}\right)$$

wherein W and W' are the weights or areas of the triangles whose centers of gravity are respectively g and g'.

distance $g\,G = g\,g'\left(\dfrac{A'_g}{A_g + A'_g}\right)$ where A_g and A'_g are the areas of

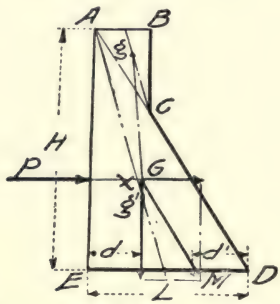


FIG. 239.

the triangles, of which they are the center of gravity. The triangle $A\ B\ C$ is added on to give a strong crest. The base $E\ D$ is arbitrarily taken as two-thirds the height H. The resultant $I\ M$ is next obtained, and from $p = W\dfrac{(L-d)}{L\,d}$ the maximum pressures are found. Second, if the pressures of the first section are within the limits of safety another section of 50 feet is added (Fig. 240), the up-stream face being given a batter of 15 feet in the 50, and the apron a batter of 40 feet in the 50. (These proportions conform to the standard sections for such dams, (see Fig. 241.)

The center of gravity of the entire figure $A\ B\ C\ D\ H\ F\ E$ is now found. The center of gravity of $E\ D\ H\ F$ being found as

in Fig. 236, and then that of the two centers of gravity G and g'', found from

$$G G' = G g'' \left(\frac{w''}{w'' + W}\right)$$

wherein w and w'' are the weight or areas of the sections whose centers of gravity are G and g'' respectively. G' is the center of gravity of the entire section.

Find P for the dam now 150 feet high and its point of application in the same way as for the 100-foot section.

As the face of the dam has now been given a batter of 15 feet

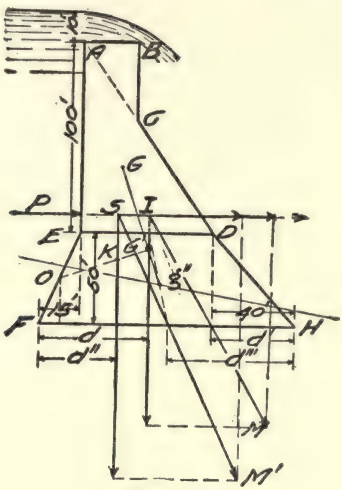


FIG. 240.

up stream, there will be a certain component of force, due to the water holding the dam down. This will be

$(100+10+\frac{50}{2})\times 62.5\times 15 = 126{,}562.5$ pounds. This will act

perpendicular to the base $F H$ and at a point half way between E and F. This large component is not generally considered in the design of the dam, and while its neglect is on the side of safety, in finding the pressure on the toe H, it adds to the pressure on the base at F, and the author believes that if there is any reliability at all in the designing of a dam, every factor should be allowed for. Therefore, to allow for this force con-

sider the 126,562.5 pounds as being so much area or weight of masonry concentrated at O, after having divided it by the weight of a cubic foot of the dam materials to reduce it to the same standard with the former calculations. Thus, if the masonry weighs 140 pounds the equivalent at O = 904.02 square feet.

Now connect O and G' and find the center of gravity of these two areas, and from the point so found, drop a perpendicular to the base $F H$ and on this new component lay off to scale the

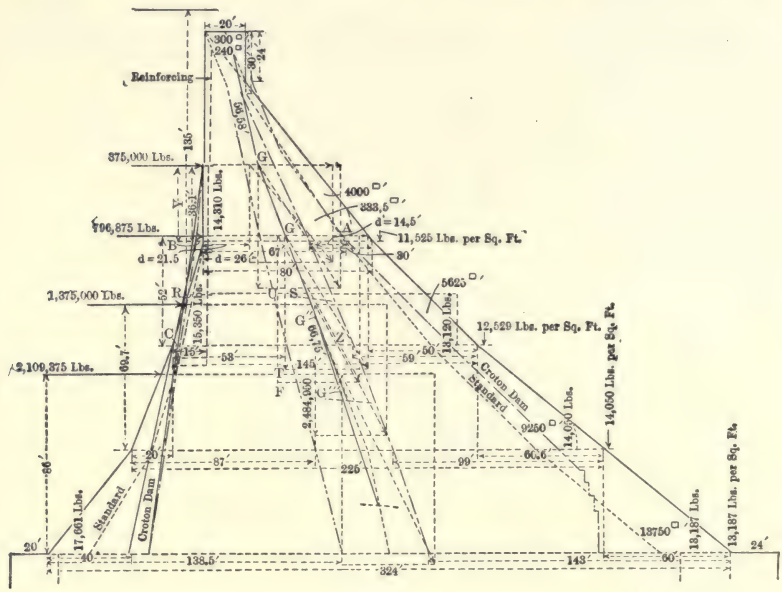


FIG. 241.

amount of the combined forces due to G' and O and draw the resultant $S M'$.

The effect has been to throw the resultant more toward the center of the base, which is as it should be. The pressure of the dam when the pond is empty may be too great at F and possibly exceed the stress allowed, therefore the slope $E F$ may have to be increased. From $p = \frac{W\ (L-d)}{L\ d}$ the pressure at F can be calculated for either d or d'' and at H for either d' or d'''.

If the pressures are satisfactory lay down another 50-foot

section and proceed as before. Each of the succeeding sections batter 50 feet down stream, up stream the third section batters 20 feet, the fourth 30 feet, the fifth 50 feet, etc. Of course these batters are arbitrary and are only to serve as a preliminary guide.

EXAMPLE.—As example, assume a dam 250 feet high, with 10 feet of water going over its crest; also assume the masonry weighs 130 pounds per cubic foot, allowing 10 pounds for flotation, and that the safe bearing strength is 20,000 for d and 15,000 for d'. Now assume the section to be the same as the standard dam (shown in Fig. 241 by the dotted outline). Then the crest is 20 feet wide and formed of a triangle 20 feet wide at base and 30 feet high superimposed on the triangle 100 feet high and with a base two-thirds the height or 66⅔ feet. The calculations for this first section follow:

First section. The center of gravity for each triangle is found and the center of gravity G of the two areas determined as explained above. Then calculate the water pressure against the dam by $P = 62.5\,H\left(\frac{H}{2}+h\right)$, which for this section is,

$$P = 62.5 \times 100\left(\frac{100}{2} + 10\right) = 375{,}000.$$

The point at which P is applied is the distance Y above the base thus,

$$Y = \frac{H}{3}\left(1 + \frac{h}{H+2\,h}\right)$$

which here is

$$Y = \frac{100}{3}\left(1 + \frac{10}{100+20}\right) = 36.1'.$$

The force P is projected horizontally as shown by the arrow heads. A perpendicular line is dropped from the center of gravity G, and projected till it intersects P. From this intersection measure, to any convenient scale, the force P, horizontally, and the weight of the section, W, downward, as shown, complete the rectangle and draw the resultant. Now measure the distance from the intersection of this resultant with the base of the dam to the right hand edge of dam, this is d' and

in this case equals 14.5′. Also measure the distance from the perpendicular from the center of gravity to the up-stream edge of the dam, $d = 21.5$ feet. Areas about $G = 300 + 3333.5 = 3633.5$. $W = (300 + 3333.5)\ 130 = 472{,}355$ pounds.

$$\text{Maximum pressure } p_{max} = \frac{W(L-d)}{L\,d}$$

and for d,

$$p_{max} = \frac{472355\ (67 - 21.5)}{67 \times 21.5} = 14{,}989 \text{ pounds per square foot.}$$

for d',

$$p_{max} = \frac{472355\ (67 - 14.5)}{67 \times 14.5} = 25{,}525 \text{ pounds per square foot.}$$

This gives a pressure on the up-stream side of dam below the assumed allowable stress of 20,000, but the pressure on the down-stream side is much too high, therefore the section is altered giving the base a width of 80 feet.

Then the area about G is $240 + 4{,}000 = 4240$.

$W = 4240 \times 130 = 551{,}200$ pounds.

$$p_{max} \text{ at } d = \frac{551{,}200\ (80 - 26)}{80 \times 26} = 14{,}310 \text{ pounds per square ft.}$$

$$p_{max} \text{ at } d' = \frac{551{,}200\ (80 - 30)}{80 \times 30} = 11{,}525 \text{ pounds per square ft.}$$

We now have the p_{max} at d' somewhat lower than the limiting value but this is necessary to keep the slope further down from becoming too flat.

In all these calculations it is assumed that the extreme edge of the down-stream toe would not crumble if the dam should turn on it as a pivot. This would not be true as it would break back to some point, A and therefore d' should only be figured to that point. This would greatly increase the p_{max}. However, the authorities do not generally allow for it.

Neither has the vertical component of the water pressure which would greatly increase the p_{max} at the up-stream edge of the dam been allowed for here.

Second section. Now add 50 feet more of dam as shown and,

$$P = 62.5\times 150\left(\frac{150}{2}+10\right) = 796{,}875 \text{ pounds.}$$

$$Y = \frac{150}{3}\left(1 + \frac{10}{150+20}\right) = 53 \text{ feet.}$$

Areas about $G' = 240+4000+5625 = 9865$ square feet.
$W = 9860\times 130 = 1{,}282{,}450$ pounds.

Completing the parallelograms of forces,
for d,

$$p_{\max} = \frac{1{,}282{,}450\ (145-53)}{145\times 53} = 15350 \text{ pounds per square foot.}$$

for d',

$$p_{\max} = \frac{1{,}282{,}450\ (145-59)}{145\times 59} = 13120 \text{ pounds per square foot.}$$

These values being sufficiently safe, another 50 feet of dam is added.

Third section. $P = 62.5\times\frac{200}{2}\ (200+10) = 1{,}375{,}000$ pounds.

$$Y = \frac{200}{3}\left(1+ \frac{10}{200+20}\right) = 69.7 \text{ feet.}$$

Areas about $G'' = 240+4000+5625+9250 = 19{,}115$ square feet.

$$W = 19{,}115\times 130 = 2{,}484{,}950 \text{ pounds.}$$

Completing the parallelogram,
for d,

$$p_{\max} = \frac{2{,}484{,}950\ (225-87)}{225\times 87} = 17510 \text{ pounds per square foot.}$$

and for d',

$$p_{\max} = \frac{2{,}484{,}950\ (225-99)}{225\times 99} = 14050 \text{ pounds per square foot.}$$

Each of these pressure seems to be approaching our assumed limits so another 50 foot section is added.

Fourth section. $P = 62.5 \times 250\ (250/2 + 10) = 2{,}109{,}375.$

$$Y = \frac{250}{3}\left(1 + \frac{10}{250 + 20}\right) = 86 \text{ feet.}$$

Areas about $G''' = 240 + 4000 + 5625 + 9250 + 13{,}750 =$ 32865 square feet.

$$W = 32865 \times 130 = 4{,}272{,}450$$

Completing the parallelogram of forces,
for d,

$$p_{\max} = \frac{4{,}272{,}450\ (324 - 138.5)}{324 \times 138.5} = 17.661 \text{ pounds per square foot.}$$

and for d',

$$p_{\max} = \frac{4{,}272{,}450\ (324 - 143)}{324 \times 143} = 13{,}187 \text{ pounds per square foot.}$$

The values assumed above for the compressive strength of the masonry are very low, but will serve to illustrate the method. If the drawing is made to a scale of ¼-inch to the foot, and carefully done, the degree of accuracy will be well within all requirements.

The crest would, of course, have to be given a different shape as explained on page 254 for an overflow dam.

The outline of a standard dam section is shown in Fig. 241. This is for a dam which allows for no overflow and which assumes a cubic foot of masonry at 140 pounds. The assumption of weight is very important and the writer thinks, that owing to the difference in opinion on the subject, this is much too high. Again the crushing strength on the up-stream side is limited by the strength of the mortar, and not the stone. Also, in finding the value for $p_{\max}$ on the up-stream side, the *maximum* weight of the masonry *must* be used because it is when the reservoir is empty, and there is no flotation, that this stress will reach a maximum. This involves the construction of a second parallelogram for each section, having W figured with the weight of masonry at 170 to 180 pounds per cubic foot.

Again, suppose the back water stands at the level of the top of last section, then the weight of this section must be figured as having lost 62.5 pounds per cubic foot of masonry at all times. Being buried in the earth does not alter the calculations, all of which are figured from solid rock bottom.

The height of such a dam is only limited by the compressive strength of the masonry (the tensile strength is not considered) and the cost. The Croton, or Quaker Bridge dam is 300 feet high at the highest point.

Examining the four sections of the preceding examples for this factor of safety we find:

$$\text{First section factor of safety} = \frac{44 \times 551,200}{36.1 \times 375,000} = 1.78.$$

$$\text{Second section factor of safety} = \frac{82 \times 1,282,450}{53 \times 796,875} = 2.48.$$

$$\text{Third section factor of safety} = \frac{124 \times 248,4950}{69.7 \times 1,375,000} = 3.21.$$

In the above we have taken 44.82 and 124 as the leverage to the probable breaking line of toe. As the factors are increasing, we need not begin the fourth section.

The above factors do not allow for the tensional strength, which is the only safe way to figure, and it therefore would seem that the first section is weak. In no other branch of engineering would such a small factor of safety be used. On the first section there is no vertical component of water pressure to add to the factor of safety, the safety of this is out of all proportion to the other sections.

In the second section if the vertical component of the water pressure is to be allowed for, proceed as follows: Vertical pressure $P = 135 \times 62.5 \times 15 = 126,562.5$ pounds. This acts at a point midway between B and C and, in effect, acts the same as so much masonry, whose center of gravity is at that point; therefore connect this point by a horizontal line with the perpendicular $G' T$, and get the center of gravity of 126,562.5 pounds at R, and 1,282,450 pounds at S, as follows:

By measurement, $R S = 46$ feet.

$$R U = 46 \frac{1,282,450}{1,282,450 + 126,562} =$$

and

$$.911 \times 46 = 41.9 \text{ feet.}$$

W now $= 1{,}282{,}450 + 126{,}562 = 1{,}409{,}012$ pounds and acts through U. Lay off from U the vertical $U\,F$ to same scale as W and construct the parallelogram of forces, P remaining the same. The resultant now strikes the base at Z and the factor

of sa is now, $\dfrac{86 \times 1{,}409{,}012}{53 \times 796{,}875} = 2.87$ instead of 2.48 as before.

The $p_{\max}$ for d is also increased for the second section.

$$p_{\max} = 1{,}409{,}012 \ \frac{(145 - 79.5)}{145 \times 49.5} = 18{,}750 \text{ pounds per square}$$

foot instead of 15,350 as before.

The $p_{\max}$ is, of course, decreased on d' but this is of no importance.

There are several methods of solving the above problem,

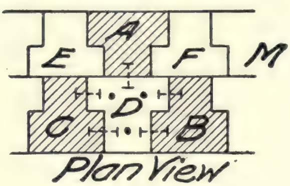


Fig. 242.

but the center of gravity method is the one most readily understood by the average engineer.

Such dams are now frequently made of concrete, and when properly made are superior to masonry. Large quantities of concrete hastily built are dangerous in any structure, but especially so for dams. Severe stresses are set up by the shrinkage of the mass on setting. The outside contracts first and later the interior, causing intense strains which result in cracks or cause the dam to give way under small added pressures.

Therefore, the concrete should be deposited in blocks as in Fig. 242. The blocks A, C and B are built in place by suitable forms and after they have set for two or three weeks, the blocks E, D and F are laid. A, B and C are lower by a foot or so than the others, so as to afford the proper friction between the layers. The blocks may be attached to each other by means of steel anchors as shown in the block D.

Steel bars placed as shown by dots on Fig. 242 will make the factor of safety what it should be; this reinforcing will be especially advisable in the case of high dams where the factor against overturning is small near the crest.

Edward Wigman, a noted authority on masonry dam construction, states the following in his admirable book: "As the theory of masonry dams has to be based upon hypotheses which are only approximately correct, we may permit," etc.

He also gives the following for the safety of a masonry dam against sliding: $f\,W =$ horizontal thrust, P, of the water. $W =$ weight of dam, and $f =$ coefficient of friction of masonry on masonry usually figured as .67 to .75.

EARTH DAMS.

The earth dam is, without doubt, the oldest form of dam, yet in this country it has been looked upon with a good deal of suspicion. Hundreds of the largest cities in the country are to-day depending on earth dams to hold the water which is to save their millions from thirst and fire. For all this when the engineer is asked to consider the building of an earth dam for a water power, there is at once a vigorous protest.

There are several very good reasons why an earth dam may be superior to the solid concrete or masonry dam, given that it be properly constructed. No sudden crack can destroy the whole structure. Any trouble which may develop will ordinarily give warning in time to permit of repairs. The Johnstown flood is pointed to as an example of the insecurity of such dams. However the cause of this disaster was due to insufficient spillways and no one claims that an earth dam is built for *spillway* purposes.

A large majority of the failures of earth dams have been due to this cause. The second in order of the causes of failure is the placing of pipes through the embankment. Build the dam so that the spillway is ample and in the proper place, place no pipes through the fill and use the proper materials and the earth dam is, without question, the best and cheapest dam that can be built.

Mr. Burr Bassell in his book, "The Earth Dam," treats the subject at length.

There have been built eleven dams over 100 feet high, ten

over 90 feet high, and six over 80 feet high. In Europe the earth dam has received considerable study and many dams are standing which are exceedingly old.

That an earth dam should be suited to a site there must be present three conditions. First, the conditions must be such that the maximum flood which has ever occurred at the place can be taken care of without washing out the uncompleted structure. Second, the conditions must be such that the maximum floods which are to be expected can be diverted around one end of the dam, over the rim of the containing cliffs or through some new channel so that at no time in the future can the waters top the embankment. Third, the materials must be suited to the work.

Mr. Burr Bassell advises that where a tunnel of sufficient size to carry all the flood water can not be cut around the dam through solid rock, or where the channel can not be entirely

FIG. 243.

changed to bring about the same degree of safety during construction, that the earth dam should not be built. This is certainly worthy advice from the strongest advocate of an earth dam, but it should be possible to use the idea shown in sketch, Fig. 243.

The author was called upon to pass on the feasibility of an earth dam near Baltimore where it was impossible to take care of the water in any other way than in the one shown in the sketch. The arches of the concrete bridge could be made of great span so as to take care of the greatest floods during construction. When the earth fills were completed a dry period would be selected and the water made to run through a comparatively small pipe until all the space under the arches was filled with concrete. Then the small pipe could be closed also or left with a gate for future use.

The best form of spillway is where the dam is of such height

that by slight excavation the water may be made to spill over the rim of the basin above the dam. The capacity of this spillway may be determined by computing the maximum flow as explained in Chapter III. Where the flood flow is small the tunnel may be resorted to.

There are certain materials which are of no use for building earth dams. It is the author's opinion that all these dangerous materials contain what is commonly called quicksand.

The test for such earths is to mix the material up fairly wet in a box and pack it with a tamp. If it quakes on being tamped it belongs to the dangerous class. Good earth should pack solidly into place.

Another test is to find at what angle the earth will stand when piled up in water. This angle should not be less than 20 degrees. The best soils for the purpose contain some clay.

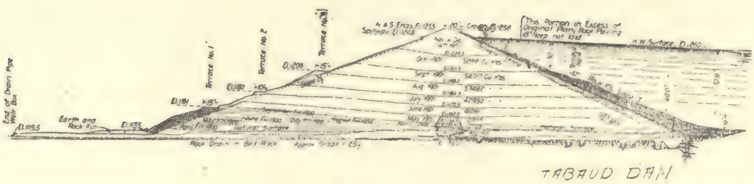


FIG. 244.

Puddle is a term given to that part of the fill which is made of selected materials mixed together to form a core to prevent seepage. Many engineers contend that the best earth dam is one of homogeneous section. Mr. Bassell placed the core of the Tabaud dam on the up-stream edge (see Fig. 244), and this is undoubtedly the proper place for it if it is used at all. Mr. Herbert M. Wilson advises the following mixture for puddle:

Coarse gravel	1.0 cubic yard
Fine gravel	.35 " "
Sand	.15 " "
Clay	.20 " "
	1.70 " "

which, when rolled in embankment, gives $1\frac{1}{4}$ yards.

All clay shrinks on drying, and if allowed to dry out while being compacted the result will be a leaky and dangerous dam.

Pure clay shrinks about five per cent. on drying. Dry clay as usually excavated will absorb one-sixth its weight of water and when perfectly dry about one-third. Some clayey mixtures while hard to excavate will run like oil when wet. The cost of well-made puddle varies between wide limits, but should not cost more than from 20 to 40 cents per cubic yard.

The safest way to test materials for imperviousness is to fill the ends of glass tubes all to the same depth with the materials and fill the tubes with water. The tube which holds the water the longest is the most impervious. Mr. Bassell advises building the dam up in layers which have a slope toward the center as shown in Fig. 245. This is to prevent the water used in compacting from going to waste over the sides of the embankment and to insure the continued dampness of the layers exposed to the air. The thinner the layers the more thoroughly can they be compacted. Six inch layers were used near the bottom of the Tabaud dam and nine inch at the top.

FIG. 245.

The process then s as follows: The earth is brought to the dam in wagons or by aerial cable and deposited in regular heaps over the proper area. Road graders drawn by six horses then spread out the piles and a sprinkling cart drawn by four horses wets it down. A five to eight ton roller passes to and fro over the wet earth and finally before the next layer is deposited, a harrow roughens up the surface. Every surface is kept wet at all times. While all the most impervious materials should be deposited on the up-stream side, no great expense should be incurred in doing so as it will be better to spend that extra amount in making the dam wider at the base.

Where the fill is made by hydraulicing it is impossible to sort out the materials, but in that case it is not necessary.

Common practice seems to make the top width about 25 feet, the up-stream slope 1:3, and the down-stream slope 1:2.

There can be no hard and fast rule, however, as the slopes depend on the angle of repose of the materials, and the width at the top is merely the factor of safety for the particular condition.

Fig. 246 shows a section of the Jerome Park Reservoir for the City of New York. Here the masonry core rests on quicksand. It would seem that in the light of the recent development of steel sheet piling, that piling driven to bed rock would be a much better arrangement.

The greatest chance for seepage is along the original surface which the dam rests on, and every precaution must be taken to break the continuity. Plowing and harrowing should be thoroughly done and all soft mud pockets cleaned out. If the bottom is earth, a row of steel sheet piling should be driven.

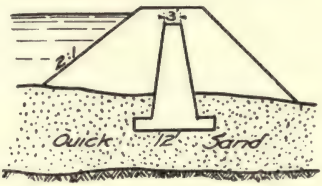


Fig. 246.

The force required to take care of and compact 2000 cubic yards per day of materials delivered on the dam will be about as follows:

Three rollers, one ten-ton and two five-ton, drawn by six horses.

Two graders, drawn by six horses.

Three water tanks, drawn by four horses.

Two harrows, drawn by two horses.

Three carts, drawn by one horse.

One plow, drawn by two horses.

The total cost of this equipment would be, exclusive of horses, about $2500 and the cost per cubic yard of the compacted materials, exclusive of the equipment, about four cents per cubic yard.

The question of drainage is one of great importance. There is sure to be more or less spring water at the site and this must all be provided with some means of exit without letting it wash out the earth composing the dam. If the bottom is of rock, trenches as in Fig. 247 should be excavated.

In the bottom of the trenches tile covered with concrete are placed. The trench thus made serves a double purpose as it not only takes care of the spring water but also acts as a preventative to seepage along the natural surface. Where the bottom is earth the spring water must be conducted away with great care.

FIG. 247.

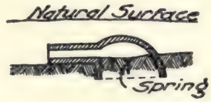


FIG. 248.

Each spring must be thoroughly boxed over with reinforced concrete and a drain built as in Fig. 248 to conduct the water away. This drain should not run at right angles to the axis of the dam, as it would then tend to produce a leak through the dam, but it should have a small angle with the axis, making

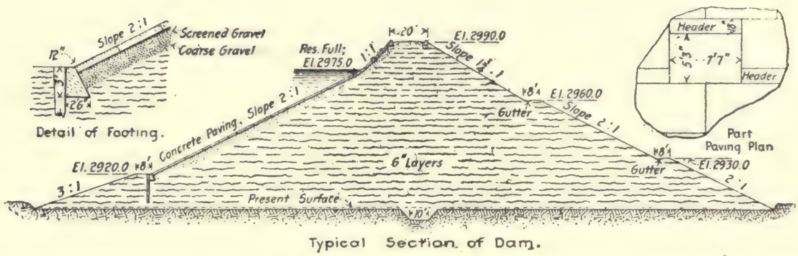
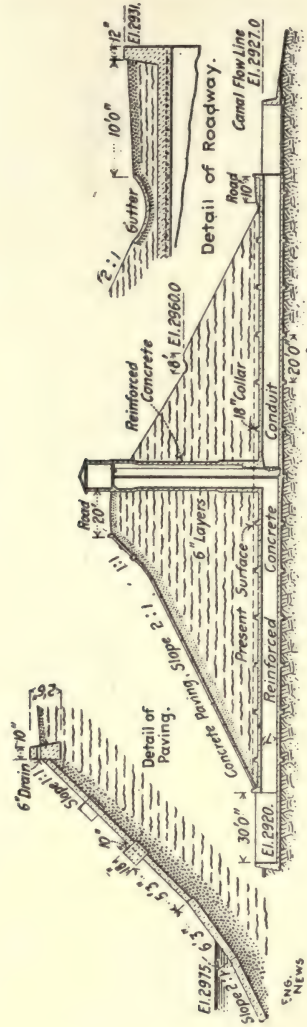


FIG. 249.—Earth dam.

as long a drain as possible before it reaches the down stream edge of the dam.

In Figs. 249 and 250 are shown sections of the Belle Fourche dam now under contract in South Dakota, the contract price for this dam containing 1,600,000 cubic yards of gumbo was at the rate of 61 cents per cubic yard (government contract), but this included spillways. The author's experience with

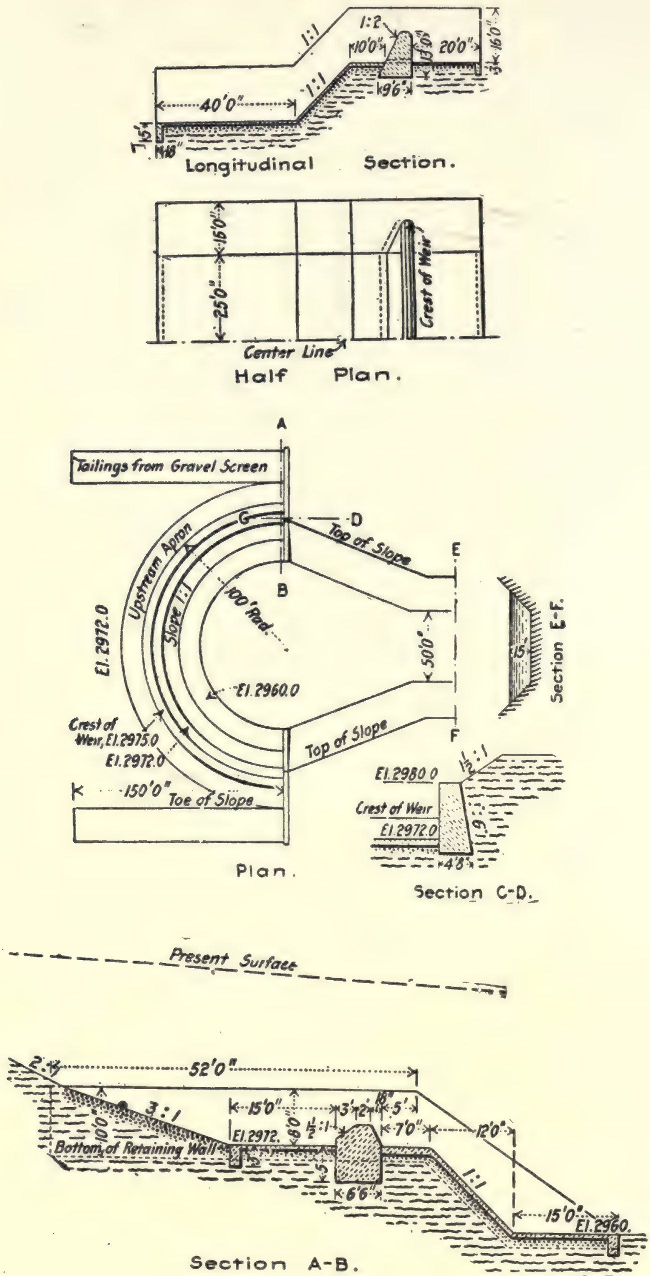
gumbo has been very disastrous and it would be considered a dangerous material for earth dams unless the slopes were at least 3:1 and the top 50 feet wide.



Section through North Conduit.

FIG. 250.—Earth dam.

Figs. 251 to 252 illustrate the weir which is to be built into the top of the earth dam. This is always a dangerous thing to do.



1:1
1:2
10'0"
13'0"
20'0"
16'0"
9'6"
40'0"
1'6"
Longitudinal Section.
16'0"
25'0"
Crest of Weir
Center Line
Half Plan.
A
Tailings from Gravel Screen
G
D
Top of Slope
E
Upstream Apron
Slope 1:1
100' Rad.
B
El. 2972.0
50'0"
15'
Section E-F.
El. 2960.0
Crest of Weir, El. 2975.0
El. 2972.0
F
Top of Slope
150'0"
Toe of Slope
El. 2980.0
Crest of Weir
El. 2972.0
4'8"
Plan.
Section C-D.
Present Surface
52'0"
3:1
10'0"
Bottom of Retaining Wall
15'0"
El. 2972.
8'0"
3'
2'
5'
7'0"
12'0"
6'6"
1:1
15'0"
El. 2960.
Section A-B.

FIGS. 251-252.

Fig. 250 shows how the concrete pipe is laid through the dam which is also bad engineering.

Hydraulic Fills.

To placer mining in the West we owe the cheapest and best method for making a fill where the local conditions are favorable.

Where a giant is used, water must be delivered to it at a head of 100 to 150 feet. In exceptional cases this water is found within reasonable distance to the materials and at a sufficient elevation to produce the necessary head. However, in the great majority of cases the water must be pumped from the river to be dammed. To pump the water a centrifugal pump may be used.

In such cases the power required for pumping is a serious

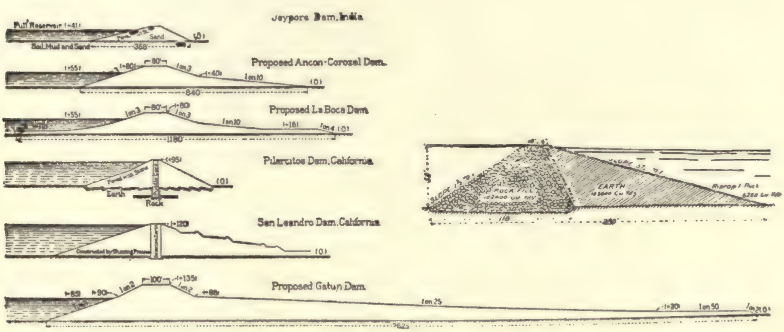


FIG. 253.—The cross sections of several of the world's most famous earth dams.

item and wherever possible the water used should be collected at the level of the giant and used over and over again. Thus a pump of comparatively small size would pump the water from the river up to the giant and a large pump at the giant would pump the pressure necessary for it.

Fig. 254 shows the method of using the giant. The grade of seven to ten per cent. from the cliff to the sluice is maintained at all times. The sluice may be a metal lined box or may be simply a ditch dug in the earth. If a metal lined box it must have a slope of from six to ten per cent., and if a ditch, 25 per cent. The sluices lead the earth and water down to the dam where the

semi-liquid is collected in a lake on top of the fill. At some suitable location a timber spillway is provided so that the surface water is drawn off without any of the earth going with it.

The amount of water used depends largely on the head at the giant and the materials, but roughly it may be taken at from 900 to 1500 cubic feet of water per cubic yard of materials in the dam.

If, as was the case with the La Mesa dam in California, there is no embankment upon which to work a giant, then a large area of soil is plowed and scraped into the sluices. At the La Mesa dam 11 acres were excavated to a depth of two feet. In this way 700 cubic yards per day were delivered to the dam with a use of 50,000 cubic feet of water.

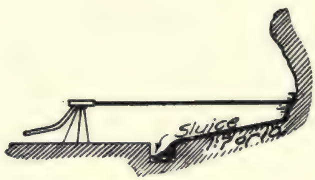


Fig. 254.—Giant washing earth into sluice.

About the largest daily average fill was made on a long railway fill where for 60 days the average was 1100 cubic yards per day.

In this case the head at the giant was 160 feet and the water used was 960 cubic feet per minute, the sluices were 4x2 feet. The costs per yard on several large fills averaged six to eight cents including all costs of plant, etc. The cost of the plant in the above instance was $10,000.

Costs.

Figs. 255-263 show graphically the cost per foot of width and amount of materials required per foot of width, for different types of dam. Figs. 255 and 256 refer to the type of dam shown in Fig. 213 and give respectively the quantity of material and cost for different heights of dam. Figs. 257 and 258 refer to the type of dam shown in Fig. 214 and give respectively the cost and the quantity of material. Figs. 259

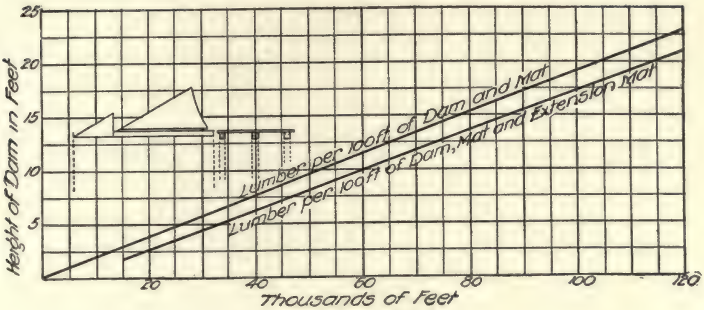


FIG. 255.

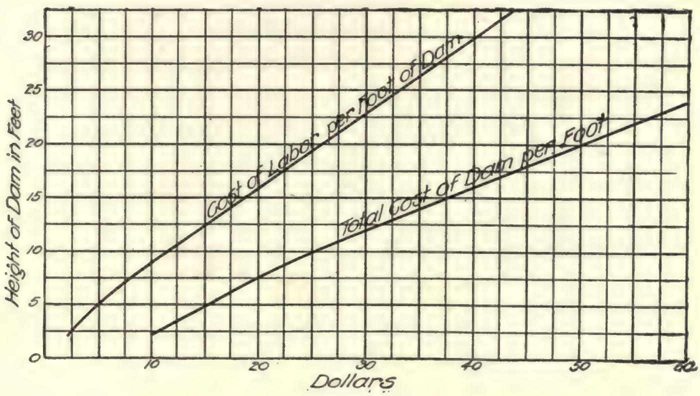


FIG. 256.

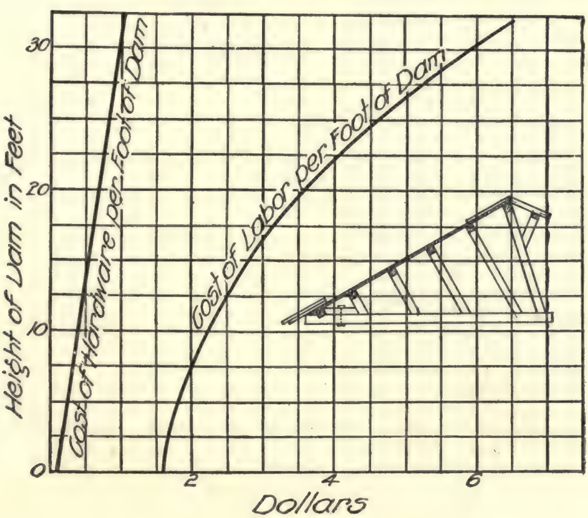


FIG. 257.

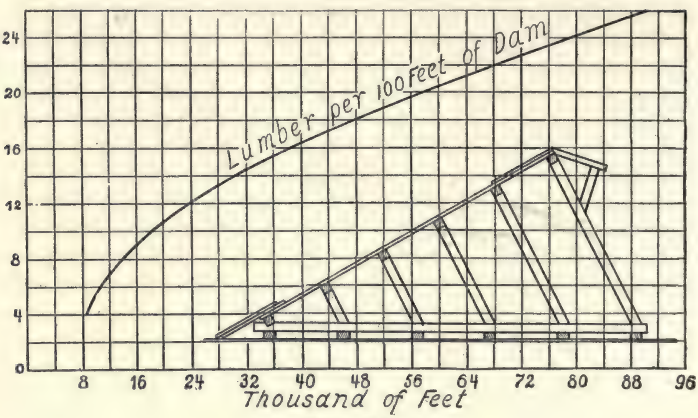


FIG. 258.

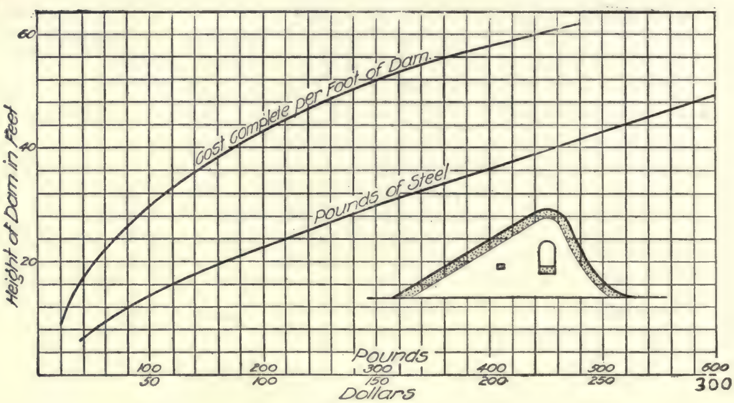


FIG. 259.

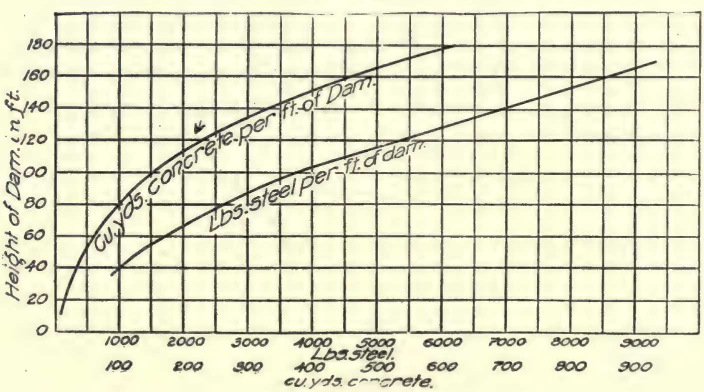


FIG. 260.

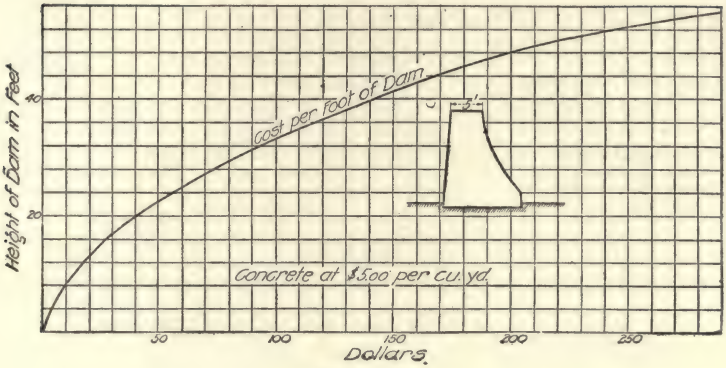


FIG. 261.

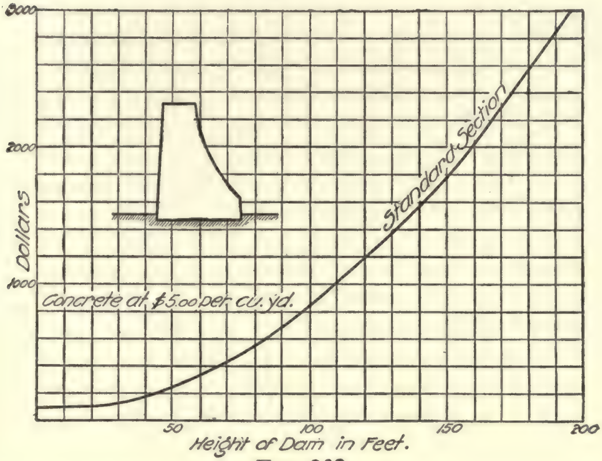


FIG. 262.

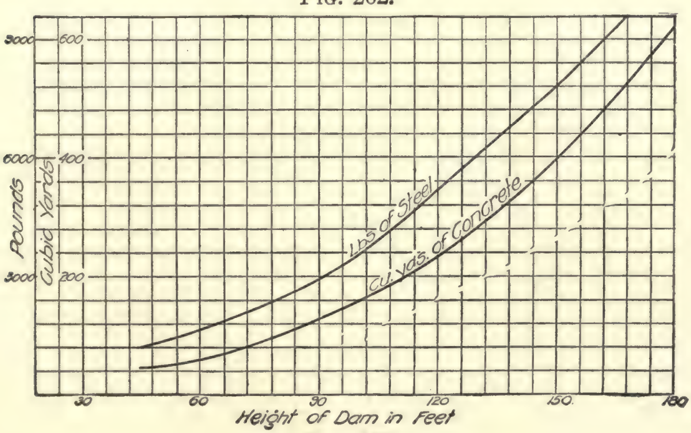


FIG. 263.

and 260 show the cost of building and the amount of steel required for reinforced concrete dams of the gravity type, built respectively for low and high heads. Figs. 261, 262, and 263 show cost data for low solid concrete, high solid concrete, and high concrete steel dams respectively. These costs do not include foundations.

ABUTMENTS.

Where the dam is on soft bottom and rests upon a mat, the abutment should in all cases stand on the same mat. The more weight on the mat the better.

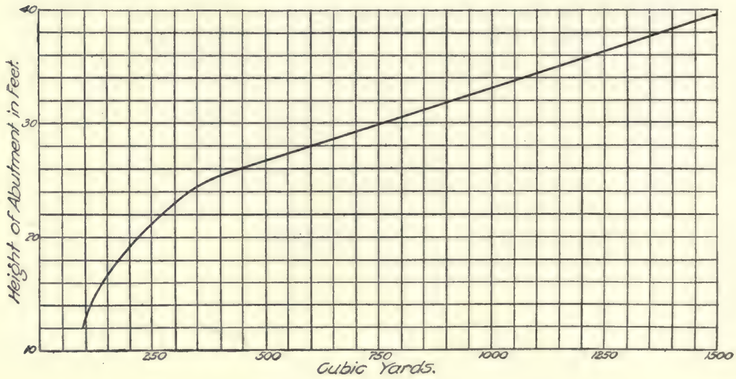


FIG 264.

The earth fill behind the abutments *must* be put in with great care. The earth should be selected and tamped in six-inch layers well wet down. Fig. 264 gives the amount of material in solid concrete abutment shown in Fig. 265.

The reinforced concrete abutment, Fig. 266, is much cheaper and better. The plan view of the solid abutment shows the wing is given a turn up stream. The object of this is to form a pocket so that any leakage is stopped by the earth falling into the corner. If a liberal wing is not provided and of this shape there will always be a leak along the abutment. The reinforced abutment has so many wings that the pocket is not necessary. Fig. 267 gives the amount of material used in the abutment shown in Fig. 266.

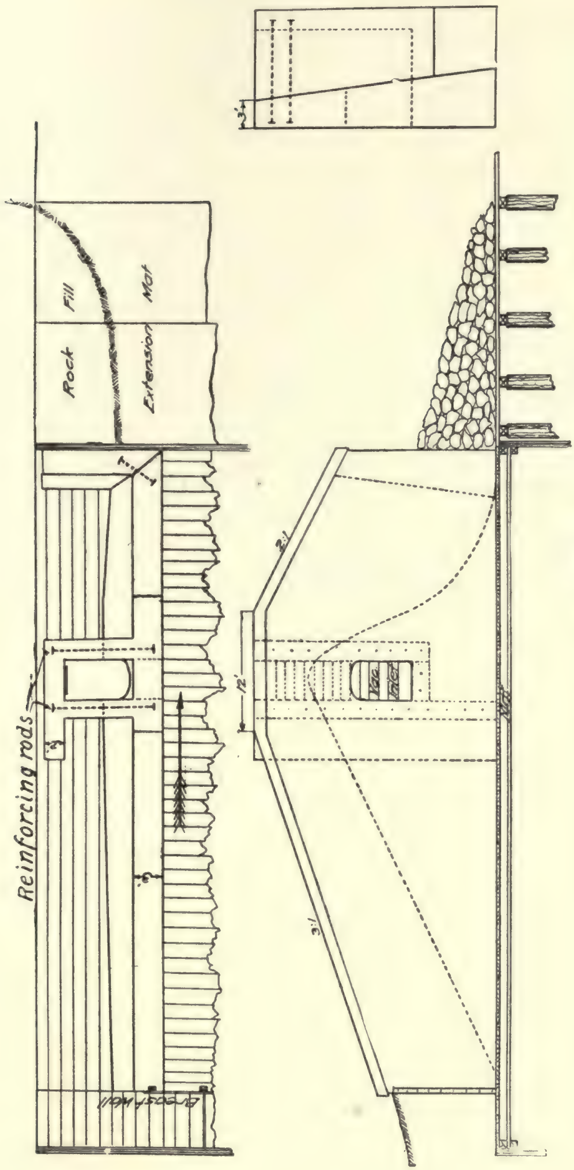


Fig. 265.

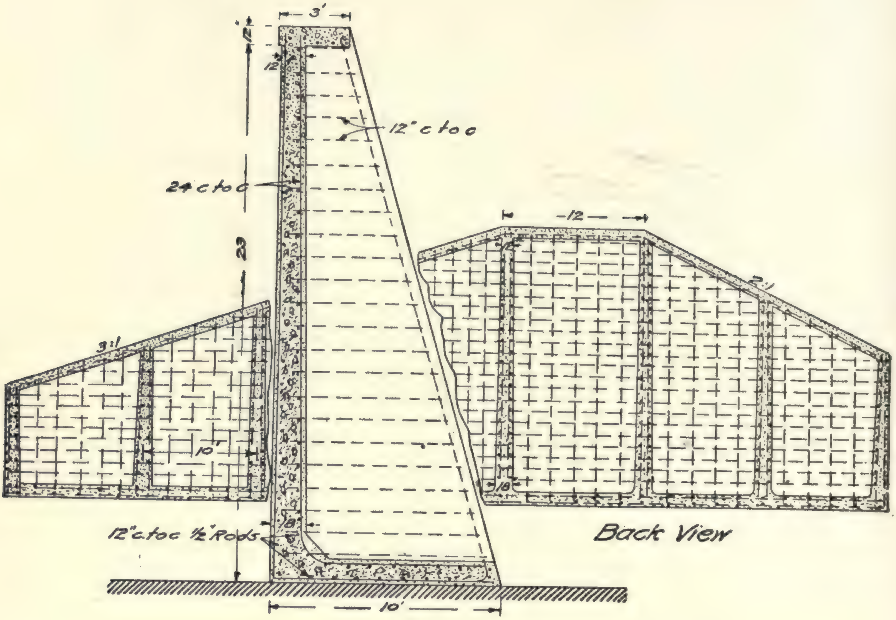


FIG. 266.

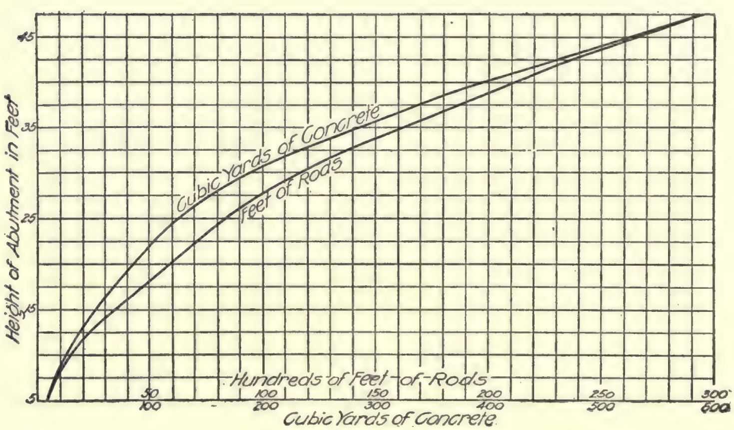


FIG. 267.

FLASHBOARDS.

The height of a dam is only limited by the value of the overflowed lands, and as it is usually only during high water that the lands are damaged, the height of the dam in summer is lower than is actually necessary. It is to increase the height of the dam during low water (just when head is most needed) that the flashboard is used. There has never been a flashboard designed that suited all requirements and yet its value is so great that most dams are equipped with some one of the various varieties.

One of the simplest forms, see Fig. 268, has proved the most satisfactory. It consists of wooden or iron posts set into the crest of the dam and supporting vertical planks against the water pressure. The posts are so designed that when the

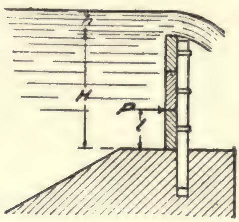


FIG. 268.

water exceeds a certain amount they break off and plank and posts go down stream. The planks are fastened to the posts by means of staples through which pass the posts, otherwise, when the water was drawn down below the crest of the dam the plank would fall off.

In *calculating the dimensions of posts*, First find the pressure against the post. $P = 62.5\,H\,(H/2+h)$ (Fig. 268), in the case where the water flows over the boards at a depth h; and $P = \frac{1}{2}\,H \times 62.5 \times H$ when it comes to some depth H, at or below the top of board. Having P, the moment of pressure against the post is found by, $P \times l$ = inch pounds $= M$. l is found where water flows over the boards by $l = \frac{H}{3}\left(1 + \frac{h}{H+2\,h}\right)$ and for water at top of boards $l = H/3$. l must be given in inches.

Having determined the permissible height of water over the

boards, find the moment of pressure for that height and then for the water at the top of the board.

The resisting moment of the post M is then found for both cases, as follows: Let s = the safe strength per square inch of the post. Then if post is of round section

$$M = .098 + s + d^3$$

See properties of sections page.

For example: A flashboard three feet high and posts of round white pine and three feet between centers with a depth of not more than 12 inches of water over the boards; find the diameter of the post.

First, when water is 12 inches deep over boards, $P = 62.5 \times 3 \left(\frac{3}{2} + 1\right) = 469$ pounds per foot length or $469 \times 3 = 1407$ pounds pressure against one post. This pressure P acts at a height of $\frac{3}{3}\left(1 + \frac{1}{3+2}\right) = 1.2$ feet or 14.4 inches from the bottom. Therefore the moment of pressure against the post is

$$M = 14.4 \times 1407 = 20{,}261 \text{ inch pounds.}$$

Second, when the water is at top of boards:

$$P = \frac{3}{2} \times 62.5 \times 3 = 281 \text{ pounds per foot}$$

or

$$281 \times 3 = 843 \text{ pounds on post.}$$

$$l = \frac{3}{3} = 1 \text{ foot} = 12 \text{ inches.}$$

The moment of pressure is

$$M' = 843 \times 12 = 10{,}116 \text{ inch pounds.}$$

The moment of posts' resistance in the first case is

$$M = .098 \times s \times d^3$$

If we take $s = 3200$ as the *breaking* strength of white pine and substitute, we have

$$.098 \times 3200\, d^3 = 20{,}261 \quad \text{and} \quad d^3 = \frac{20{,}261}{3200.098}; \, d = 4. \text{ in.}$$

With water at top of board $3200\, d^3 \times .098 = 10{,}116$; $d = 3.2$ in.

inches. Therefore, the post should be slightly more than 3 inches in diameter. From this it is seen that the posts will be safe with water at top of the boards, but will break when the water is a few inches above, as Table **53** gives the breaking strength of white pine as 4000 pounds square inch. For square posts

$$\frac{s\ h\ d^2}{6} = P\,l.$$

After a few seasons of actual test the exact size of the posts will have been determined.

The loss of the boards in most cases is of small moment as the dam is only a few hundred feet long, and in many cases the plank can be caught below. However, the posts do not always break at the right time, often breaking too soon and causing a waste of water. They may also concentrate the current where a few posts have broken prematurely, thus causing uneven wear on the dam crest.

In the plans for the Yorktown dam Fig. 300 is shown a form of flashboard designed by the author. It is especially adapted to short dams, though it can be applied to any length. By turning the shaft all the posts are lowered or raised at the same time. In lowering, however, as the top of the posts get below the center of the first plank, that plank tips over and goes down stream. As the tops come to the centers of the next plank they also pass over and so on with as many planks as the boards are high. After the flood has subsided it is an easy matter for a man to walk out on the crest and place new boards.

Such flashboards are seldom over three feet high though they have been built four feet.

Quite a severe vacuum forms under the water sheet and unless air inlets are provided the posts will fail much sooner than the above calculations would indicate.

A form of flashboard patented by J. H. Shedd and O. P. Sarle is shown in Figs. 269 to 270. These were used at Norwich, Conn., on a dam 432 feet long and have apparently given satisfaction

Referring to Fig. 270 the board *m* is pivoted on a toothed cam roller *e*. The object of *e* is to vary the center of resistance to suit the varying center of water pressure due to an increasing

flow of water over the boards. With the water at the top the center of water pressure is slightly below the teeth then in contact. As the flood comes on, the center of pressure

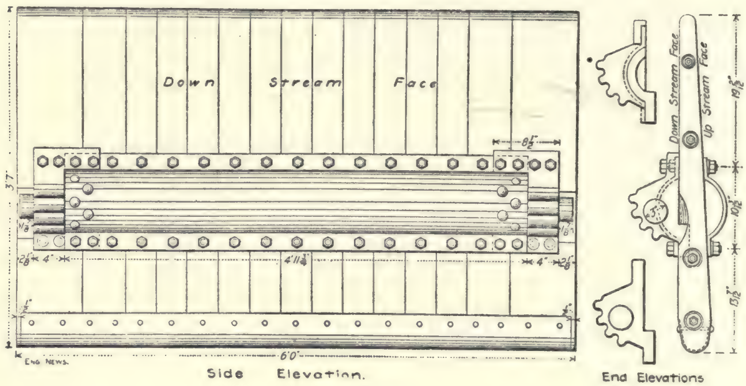


Fig. 269.

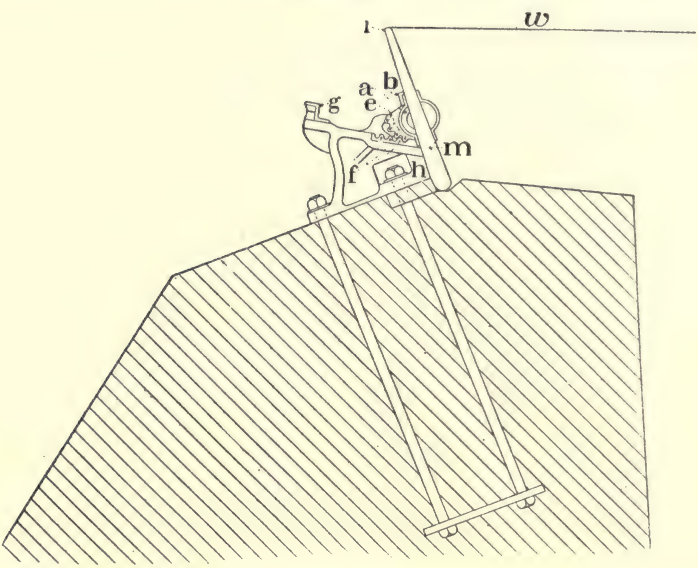


Fig. 270.

rises and gradually tilts the gate until when the water has reached a certain stage it lies in a horizontal position resting on the *Z* bar *g*, which runs full length of the dam. As the

flood recedes the boards automatically resume their normal position. Fig. 270 is a view of the entire flashboard mounted on the dam, and Fig. 269 a detail of one section of flashboard.

While possessing some very valuable features this form has certain defects. There are a great many obstructions for drift wood to strike against and lodge upon. It gives the water a perpendicular fall upon the dam, causing heavy vibrations. There is a serious liability to damage from ice floes and heavy logs. All the sections do not assume the same relative position at all stages of flood water. There is some leakage between the sections, etc.

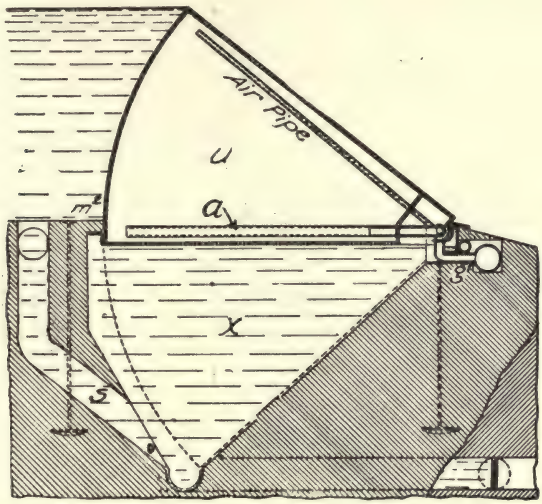


FIG. 272.

Fig. 272 shows a mechanism invented by S. C. Irwan and A. M. Bournan which, though more especially meant for a movable dam, is well adapted for flashboards. The part u is in the form of the sector of a circle, is hollow and extends either the full length of the dam or in long sections. The sector u floats upon the water admitted into the chamber x when it is wished to elevate the crest, any water contained within u being pumped or drained out through the pipe a. To lower the crest, water is let out of x and pumped or admitted into u causing u to settle down. This form of flashboard may be made of any

height, the limitations being the permissible widening of the crest of dam and the factor of safety of the dam under the increased head. The top slope of the sector affords a good spillway for the water and there are no projections for the lodgment of trash or to cause vibrations. The joints can all be made practically water-tight and the entire mechanism may be raised or lowered at any time from the shore by merely operating the air and water valves.

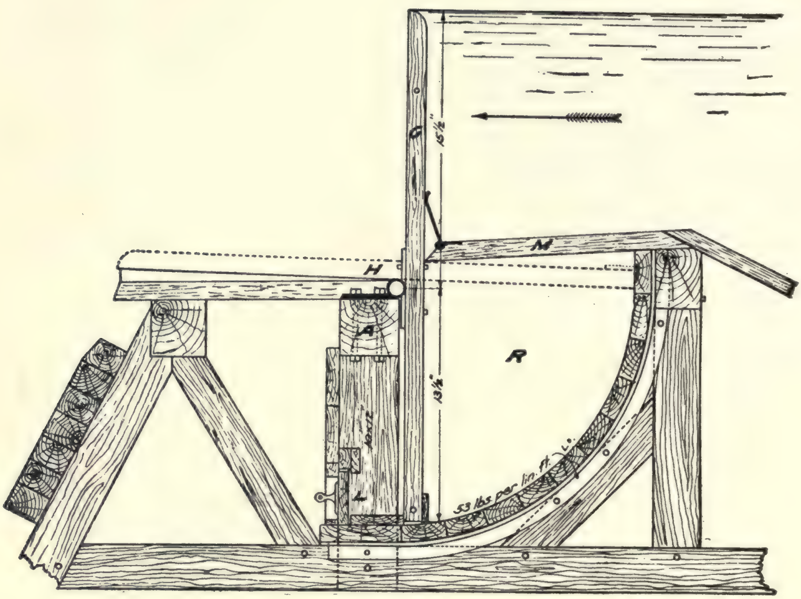


FIG. 273.

The water pressure against this form of flashboard is horizontal, and in applying it to dams must be allowed for in the design of the structure.

Fig. 273 shows a form of flashboard which possesses the controlling features of Fig. 270 and the principal of Fig. 272. Fig. 273 shows it adapted to the timber dam, and this particular board raised the water three feet. The board C is all in one length and bolted together with one-inch bolts. It is pivoted on the bronze trunions H. Every four feet along the crest is a bearing which bears on the pivot H. The pivot is supported

by the timber A. The deck planks M have a small crack left between them so that the water pressure has free access to the chamber R. About 20 feet apart are the valves L which are operated by means of a steel cable from the shore. With the valves shut the gate is as shown, but when these are open the water pressure on the left hand side of the board causes it to assume the position shown dotted. During ice flows or very high water the board is let down leaving a clear crest for the passage. When up this flashboard will leak no water, and when down leaves nothing for objects to strike against or lodge upon.

Fig. 222 shows the same type of flashboard used on a masonry dam. Spaced at intervals across the length of dam are beams A pivoted on a shaft or trunion B. Riveted to the top of these

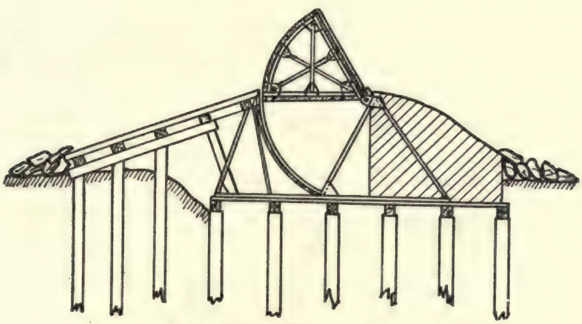


FIG. 274.—Drum Dam.

beams is the wood or steel decking. A steel plate C bent to the proper circle is attached to angle-iron imbedded in the masonry. D is a floor covering the chamber E, its only purpose being to exclude stones, sticks, etc., which might interfere with the operation of the leaf A. It is provided with sufficient openings to supply the chamber with water. The operation then is as follows: The water pressure has free access to the entire surface of the movable deck and, with the water even with the crest G, the center of this pressure P is one-third the depth from the edge F to the surface and strikes the deck at P. As the water rises, P also rises till with the water six feet above the level H, it arrives at P'. P' travels rapidly along toward G, after it passes B, permitting of very accurate adjustment. By making

F B may be more nearly horizontal if for mechanical reasons it is so desired.

If it is desired to control the movement from the shore the web *H*, the Tee-iron *I* and valve *N* are added, the web and Tee-iron making a more or less perfect shut-off of the water, so that by pumping water into the space *L* under the deck, the pressure *P* is reduced to zero and the crest will fall. If necessary, by pumps, the pressure in *L* may be made to exceed *P*.

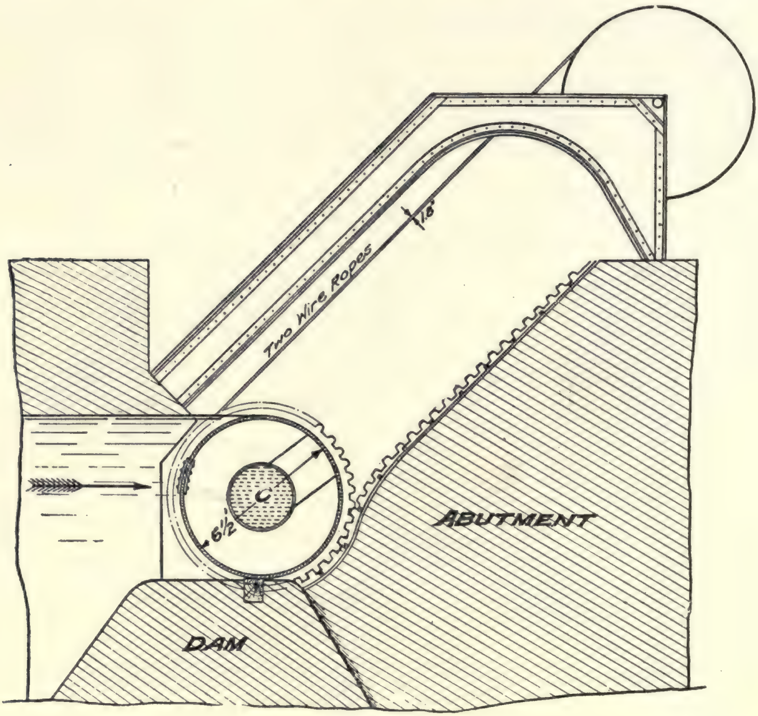


Fig. 275.

If thought necessary, a steel apron *M* (shown dotted) may be hinged to the crest to form a spillway.

This mechanism is suited to almost any range of variation in water levels, and may serve equally well as a movable dam. It may also be applied to the gravity dam as the component of the water pressure may be made to act as nearly vertical as desired. In the design shown the range of movement was

3½ feet and necessitated a width of crest of 19 feet. By making the beam *A* straight the crest could be narrowed somewhat. The vacuum which forms underneath at *O* will help in actuating the leaf since for a thin sheet of water over the crest it will be slight and for a heavy overpour quite severe. By controlling this vaccuum with suitable inlet pipes it may be increased when it is desired to lower the crest, thus sucking it downward. A float may be used to open the vacuum inlet automatically at a certain stage of the water, and if necessary, start the pumps.

Fig. 275 illustrates what is perhaps the most unique and heavy of all movable dams or flashboards. It was built in the city of Schweinfurt, on the river Main, Germany. At this place the law forbade any structure being placed in the river bed above a certain height and it was to gain the permit, that this mechanism was designed. An experimental dam was put in operation having a cylinder 13½ feet in diameter and 59 feet long and it gave perfect satisfaction. The one shown is 6½ feet in diameter and 114 feet long. The steel shell is 1.1 inch thick and weighs 193,600 pounds. This clyinder is lifted to a height of 13 feet by means of an 18 horse-power electric motor and may also be operated by 12 men in which case it takes three hours to lift.

The concentric cylinder *C* is filled with water when the cylinder is down to give added weight, but as it turns in raising this water spills out.

The lifting is all done at one end but at each end there is a cog wheel and rack.

Head Gates.

What the safety valve is to the steam engine, the head gate is to the water power. Every water power should possess such a safety device, for there comes a time in the history of every water power when it is desired to empty the head race, flumes, etc., of water and it is at such times that a reliable head gate is wanted. As the office of the head gate is to protect, it should be placed at the very entrance of the head race.

The form shown in Fig. 276 is one of the most common and we think the best. The fill *a* is made wide enough to form a roadway and heavy enough to resist the overturning force of the water. When the bottom is soft, this form of head gate

is placed upon a mat the same as a gravity dam. In nearly every case, one row of sheet piling should be driven along the up-stream edge of the mat used under head gates. The length x should generally be about equal to the depth of the water. The ends of the head gates should be well protected against the water cutting around and the down-stream wings (Fig. 276) should be extended well down stream to prevent the wash of the water after it has passed through the gates.

A velocity of 200 to 300 feet per minute may be allowed to the gates as the up and down stream length is so small that the loss of head will be inappreciable. Where the banks are a sandy loam, or other easily washed material, every joint must be water tight. A good plan is to sheet all wooden bulkheads

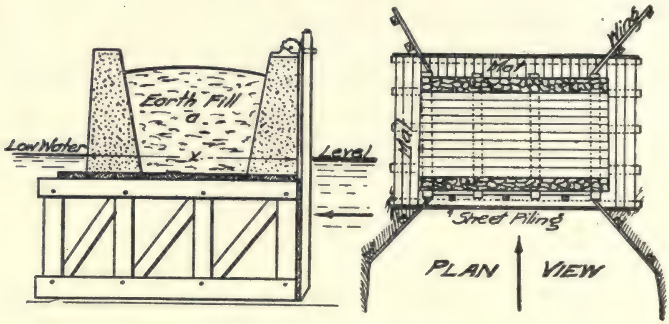


FIG. 276.—Head gates.

exposed to hydraulic pressure with "all heart" yellow pine flooring. In the last described head gate, all parts above water and for a foot or so below should be of concrete or masonry as the wood at the water line decays rapidly. The piers can also be of masonry or concrete in which case greater permanence is secured.

On large work the head gate frequently reaches heavy proportions. Fig. 277 gives a design for a gate 20x50 feet. In this design the gate proper weighs about 66,500 pounds, and to balance this, a counter balance A is used. The counter balance consists of a box girder 50 feet long and having a space 24 inches by 42 inches by 50 feet inside, which is filled with concrete, thus giving the necessary 66,500 pounds. The chain at each end of gate passes over a wheel B, the links fitting into the rim similar

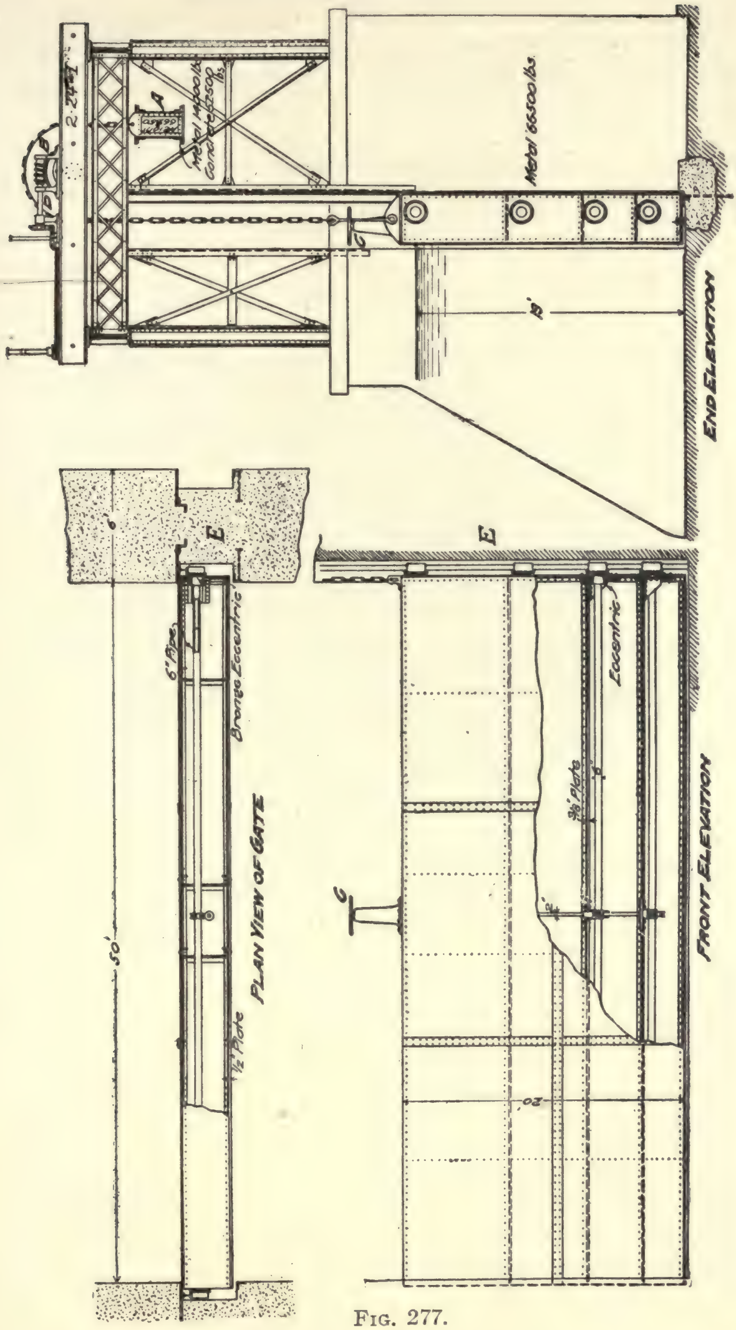

END ELEVATION
Metal 66500 lbs.
FRONT ELEVATION
Eccentric
PLAN VIEW OF GATE
Bronze Eccentric
6" Pipe
½" Plate
50'
20'
6'
E
G

FIG. 277.

to a chain hoist. To operate the gate the hand wheel C is turned until the eccentrics lift the gate away from the bearing on the pier. This throws the pressure on the eight wheels and makes

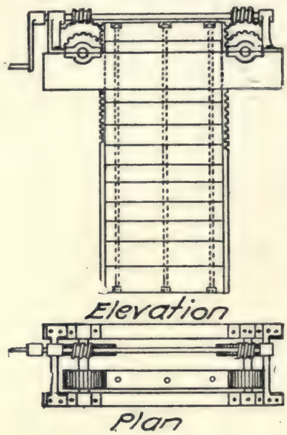


FIG. 278.

the lifting of the gate by means of the worm gear D, an easy matter.

The horizontal down stream thrust on the gate is about

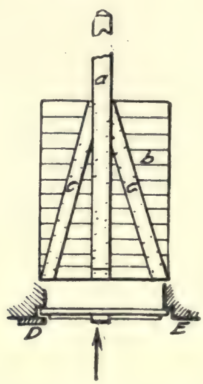


FIG. 279.—Common head gates.

630,000 pounds. Therefore the piers E must weigh 4/3 of 630,000 or 840,000 pounds to be in equilibrium against sliding and twice that for a factor of safety of two. In the design given

this factor is attained. In almost all cases instead of having some means of throwing the pressure on the wheels, a small gate would be provided to let in the water slowly and thus take off the pressure, but sometimes this is not desired as when it is necessary to control the water and limit the amount supplied to the canal. On the Chicago drainage canal and the great power at St. Mary's river, gates similar to the one shown are

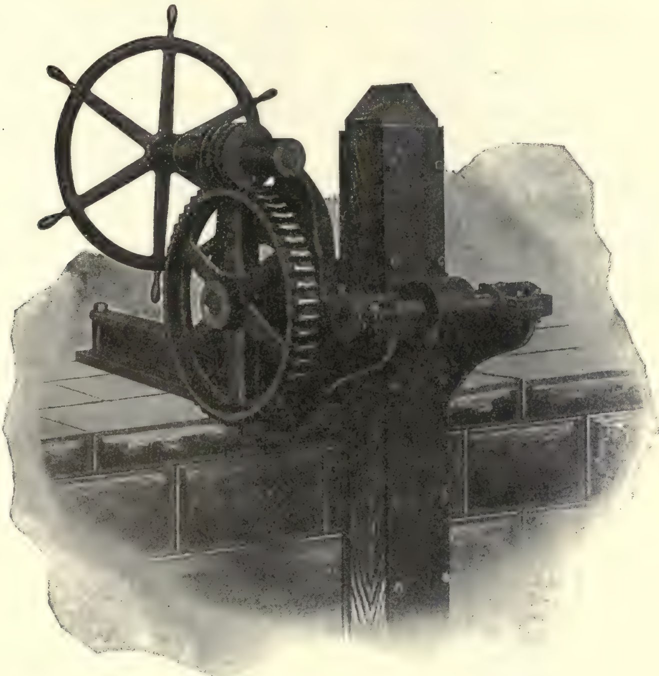


FIG. 280.

used for this purpose. The cost of this gate would be about \$8000 all complete.

Fig. 278 shows a common type of head gate suited to heads up to 20 feet. Fig. 279 shows the most common head gate and one which is plenty good enough for all ordinary circumstances. The stem *a* should be made of an 8x8-inch or a 6x8-inch timber. The planks *e* should be seasoned and edged. The braces *c* are nailed on and the stem is bolted with $\frac{5}{8}$-inch

carriage bolts. The guides for the gates should be as at E and not as at D on account of the liability of weeds and sticks getting caught and causing the gate to bind. This gate may have two stems where the area is great or the head high.

The hoists shown in Figs. 280 to 282 are suited to this gate; Fig. 282 being used for all the gates of a series except the one used to raise while the full pressure is on.

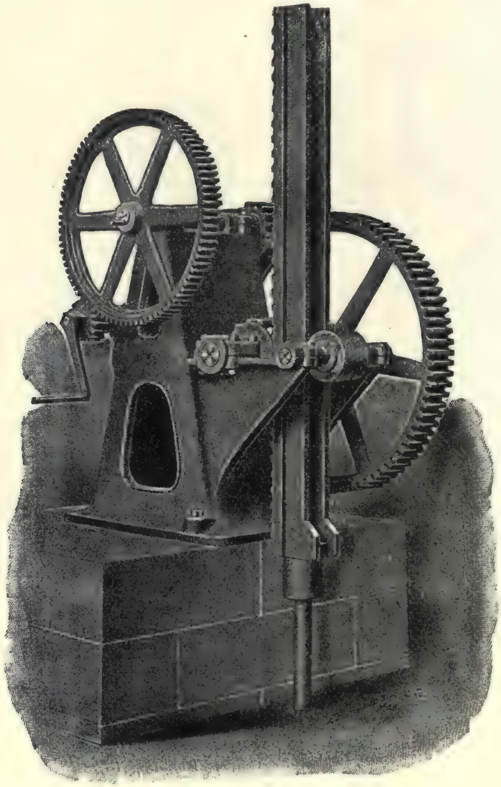


Fig. 281.

For gates under high heads (40 to 100 feet), the type shown in Fig. 283 is used. For larger gates and heads of 30 to 50 feet the hoist shown in Fig. 280 may be employed.

The elements of the design of gates is given below. The force necessary to lift is found as follows: According to usual practice the friction between oak and iron is about 62

per cent. That is, it will take 62 per cent. of the water pressure against the gate to keep the gate moving. Assuming for example a bronze gate $2\frac{1}{2}$ feet by $2\frac{1}{2}$ feet working on bronze guides under a head of 30 feet, the horizontal pressure will be $62.5 \times 29 \times 6.25 = 11{,}325$ pounds; 25 per cent. of this (for

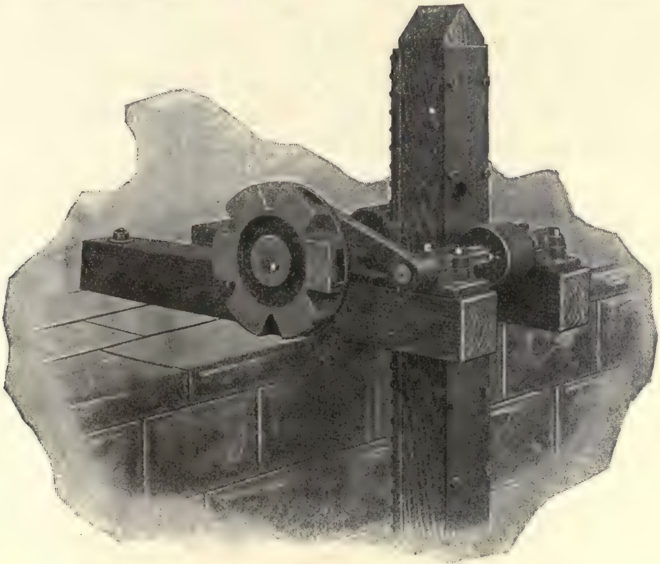


FIG. 282.

bronze on bronze) = 2831 pounds, which is the force required to move the gate. General Morin states that it requires about one-eighth more force to *start* the gate, therefore to be safe, 3200 pounds will be taken as the starting force. Now suppose

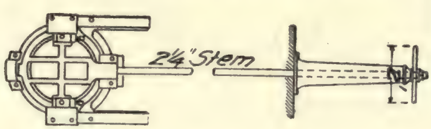


FIG. 283.

the gate weighs..................................	500	pounds
Friction of screw = $(500+7021)\times$coefficient .25...	1880	"
Equivalent weight of gate due to friction $11{,}325\times$ .25..................................	3200	"
Friction of nut = $(500+7021)\times$coefficient .15....	1128	"
Total equivalent weight, $W=$..............	6708	"

Now for the hoist shown in Fig. 283 when the hand wheel is 24 inches in diameter and the pitch of the screw on the stem is 1 inch, $W = \frac{F \times 2\ \pi\ R}{P}$ where $W = 6708$; F = force required to start gate; R = radius of hand wheel = 12 inches; $P = 1$.

Substituting, $F = 890$ pounds.

This is too large for a hand wheel and such a gate would be lifted with two such screws or a set of gears would be used to reduce the required pressure, as in Fig. 280.

Sluice Gates.

In Fig. 284 a very common form of umbrella waste gate is

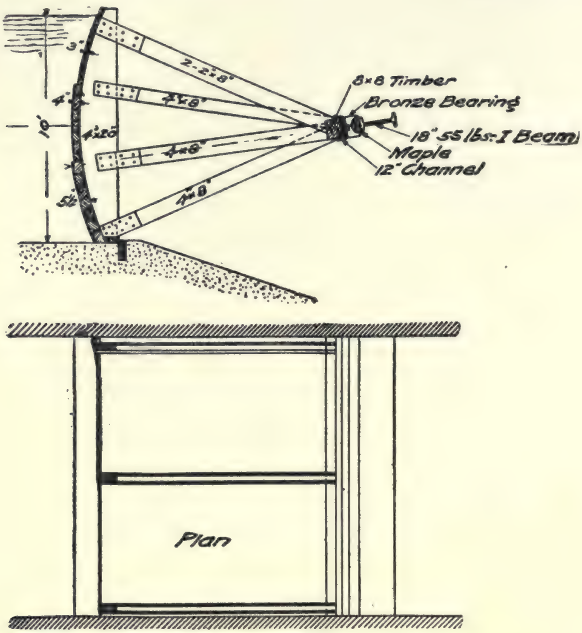


Fig. 284.

shown. This gate is often made a part of the dam. The size shown is that usually selected, there being as many gates distributed along as thought necessary to pass the water. As here shown, the down stream thrust of the water is horizontal and heavy masonry walls are necessary to resist it. The plan shown

in the Noblesville plant (Fig. 352) throws most of the pressure in a vertical direction. Experiments made by the author on old head gates show that F is about 85 per cent. of the horizontal pressure against the gate.

Fig. 285 shows a hydraulically operated gate. This gate is about the best of all the types where some form of power is available at the power house at all times. Usually when it is required

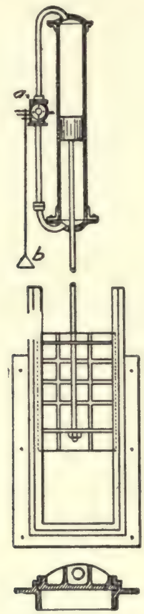


FIG. 285.

to operate the gates it is for the purpose of a shut-down, and therefore the power ceases. When there is a storage battery plant or where current can be brought to the power house from some other plant, a motor can be used to drive a force pump. Pressures of 500 to 1000 pounds per square inch should be used. The piston rod will not require packing or piston rings as some waste of water during the lift will do no harm. The valve *a* lets the water in on one side and out on the other, and by pulling on the cord *b*, the water will be admitted to either side of the piston.

Head Racks.

As the uninterrupted working of the plant depends largely on keeping driftwood and other objectionable trash out of the turbines, it becomes of the first importance to properly construct the racks. In a large majority of cases, their area is made much too small, due allowance not being made for the area occupied by the rack bars. The net area of the rack should be such that not more than 40 to 60 cubic feet of water per minute will pass per square foot of area. The cheapest form of rack bars are those made as shown in Fig. 286.

The bars *a* should be from one to three inches apart depending on the size of turbine they are to protect and should be built in sections of say, from six to eight bars held together by ½-inch

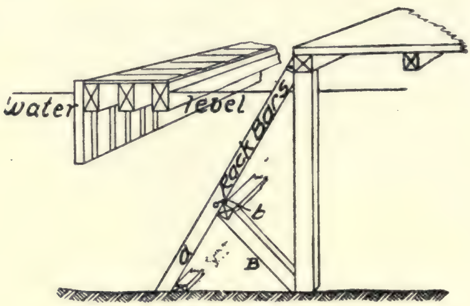


Fig. 286.

bolts *b*. The sections are not fastened to the rack frame and so may be easily removed for repairs. If the length of the bars is not over ten feet the brace *B* is unnecessary. Iron is much the best material for the bars, and, though the first cost is somewhat more than for wood, it will be found the cheapest in the long run. The bars may be of iron from ¼x2 inches to ⅜x4 inches. For heads of from 6 to 14 feet the use of ⅜x3-inch iron strengthened every six feet of its length by a ½-inch bolt, is advised, a piece of ⅝-inch gas pipe should be strung on between each pair of bars for spacers.

There should be two racks, one a coarse rack above the other. The coarse rack should have 3-inch spaces between the bars and the fine rack 1 inch to 1½ inch. For small turbines under 15 inches a brass wire screen affords excellent protection. The

meshes should be $\frac{3}{8}$, $\frac{5}{8}$, or $\frac{3}{4}$ inch square. Instead of the second coarse rack a deflecting boom, *a*, may be used (Figs. 286 and 287).

Several buoyant timbers are strung together across the head race as shown, the bars being given a slant towards the dam and projecting down into the water several feet. Such a boom will catch a large proportion of the trash and most of it will glance off and pass over the dam.

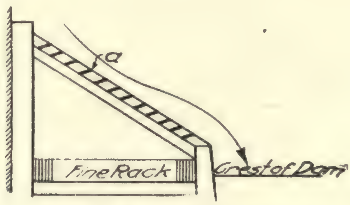


FIG. 287.

TABLE XXXVIII.

WEIGHT OF ONE SQUARE FOOT OF RACK.

Size of Bar and Weight of One Bar per Foot.		Distance Between Bars.								
		$\frac{3}{4}''$	1″	$1\frac{1}{4}''$	$1\frac{1}{2}''$	$1\frac{3}{4}''$	2″	$2\frac{1}{2}''$	3″	4″
$\frac{1}{4}''$ x 3″	2.55 lbs.	30.6	24.5	20.5	17.5	15.3	13.6			
$\frac{3}{8}''$ x 3″	3.83 lbs.	41.	33.4	28.3	24.5	21.7	19.5	16.		
$\frac{1}{2}''$ x 4″	6.8 lbs.				40.8	36.25	30.	27.25	23.4	18

Rack bars cost about three cents per pound.

Anchor ice is the worst foe to the head rack and has been known to render a valuable water power practically useless during the winter months. The cause of anchor ice is a much disputed question, but it is the belief of the writer that it is formed by the coagulation, as it were, of the water as it passes from a pool of comparatively quiet water over a shoal or rapid. When water freezes on the pond, a certain amount of the air is imprisoned in the ice making its specific gravity less than that of water but when the water has been quiet and the temperature below freezing, the water assumes a temperature below freezing and yet does not congeal until it is agitated on the rapids where it instantly freezes and contracts into solid pasty ice slightly heavier than water, so that it drifts along at all

depths and finally lodges against the racks much to the disgust of the power user.

However, if proper precautions are taken, shut-downs will never be found necessary on that account. At Norfolk, on Raquette River, the Remington Paper Co. overcame the difficulty, which at first seemed serious, by building a house over the rack and keeping it warm. Men were also kept at work with rack hooks cleaning out the anchor ice. In Fig. 288 a sketch of a patent rack is shown which gives complete protection against every head rack ill.

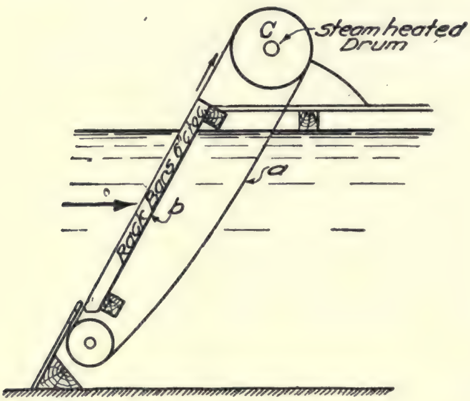


FIG. 288.

The rack is made in 6-foot sections. The wire netting *a* is such as is used for reinforcing concrete and is caused to run over the steel rack bars *b* by means of the friction drum *C*. The netting may be made to run as slowly as desired and only runs when it is found necessary to clear the rack.

A rack similar to this was used at the Mill Creek power, only it was caused to revolve by means of a current-wheel placed behind the rack and in the penstock.

CHAPTER VI.

POWER HOUSE CONSTRUCTION.

Together with head gates, racks, penstocks, etc., the accessories to the power house have been treated, some of them being at times a part of it. The power house proper, however, is the subject of this chapter.

FOUNDATIONS.

There is no one thing more important in all building operations than the foundations. After the soundings have been properly made, it should be an easy matter to design the foundations, and yet how few heavy buildings are there which do not

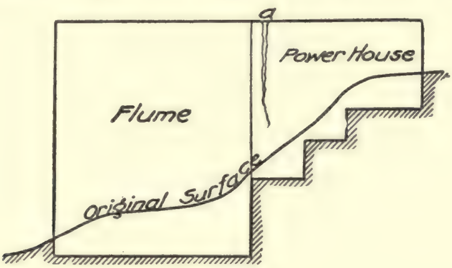


FIG. 289.

settle and show cracks. Most of the skyscrapers of Chicago are steadily sinking, as are also those of New Orleans. This is to be expected on such foundation material as those cities have, but the architect shows his skill by so designing the foundations that a 20-story building always remains plumb no matter how much it may settle. Building a power house on the bank of a river introduces a little problem in foundations.

Fig. 289 shows a power plant the flume of which rests on the solid hardpan eight feet below tail water. The power house containing the generators extends back on to the bank and rests on gravel. Also see Fig. 47.

Now, if special provision is not made to give the base of the power house foundations sufficient area, there will be a crack at *a* as shown. The flume will not settle but the generator house will.

Abutments resting on mats, and heavy wing walls running

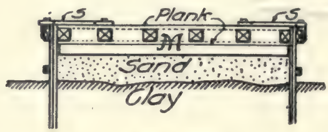
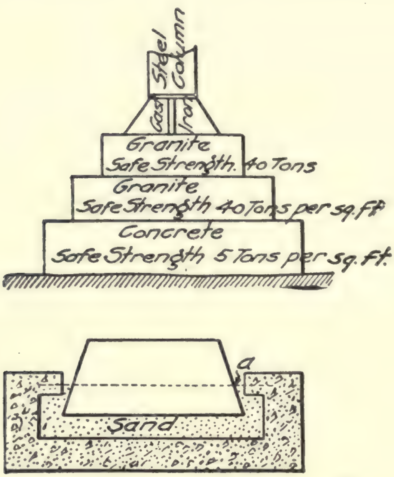


FIG. 290.

back into the shore are also subject to cracks unless carefully designed. Safe pressures for foundations are given on page 124. *All soils will settle some*, therefore the pressures must be met with sufficient bearing surface in the foundations.



FIGS. 291, 292.

For heavy concrete power houses four feet of firm soil under the base is sufficient, especially when placed on a mat as in Fig. 290. Wet sand will sustain almost any load if it is held in place by sheet piling. The mat *M* should make a close fit with the piling but not be fastened to it. In this case the

strata of clay need only be about a foot thick as the pressures are evenly distributed over it by the sand above. To prevent the piling from spreading, strap iron anchors, S, are bolted to the wales and to the mat as shown. This permits settling of the mat.

Where heavy loads are to be borne on columns the arrangement shown in Fig. 291 may be employed. The concrete must be of rich mixture.

For engine foundations where it is desired to avoid all vibrations the pier is placed on a bed of clean dry sand. As long as no sand is allowed to escape the pier will not settle, and to make this sure a copper sheet a should be embedded as shown, Fig. 292.

The heavier the foundations the less the vibration. About 300 pounds of foundation per one horse-power is good practice for engines up to 25 horse-power, 200 pounds 25 to 100 horse-power, and 175 pounds for those of 100 to 500 horse-power. Of course this only applies to foundations on soft shaky soils. The safe bearing pressures per square foot of the soil must not be exceeded (see Table XXXI), and this often makes a foundation of much larger base necessary.

Structure.

There should be only one type of power house and that type the best, but unfortunately the majority of the power plants must be built as cheaply as possible.

The flume is that part which contains the turbine. The cheapest arrangement is that in which the flume is a part of the dam. Figs. 293 and 294 show two views of such a flume, built into a gravity dam. In building this flume it must be remembered that a part of the gravity virtue of the dam is taken away and a horizontal down stream push added, therefore caution must be used in bracing the down stream bulkhead. The plan shown is for a 30-inch turbine taking 5000 cubic feet of water per minute. If the water is drawn one foot below the crest of the dam the velocity in the flume would be 90 feet per minute, which is good practice. With 12 feet of water in the flume the horizontal pressure at x = 35,500 pounds. A rod $1\frac{1}{2}$ inches in diameter will hold this and as one such rod is a small item of cost, its use is advised. Frequently

the down stream sill *A* splits along the line of mortises when a rod is not used. This arrangement could be used for two turbines in line with the dam, though it would be better to place

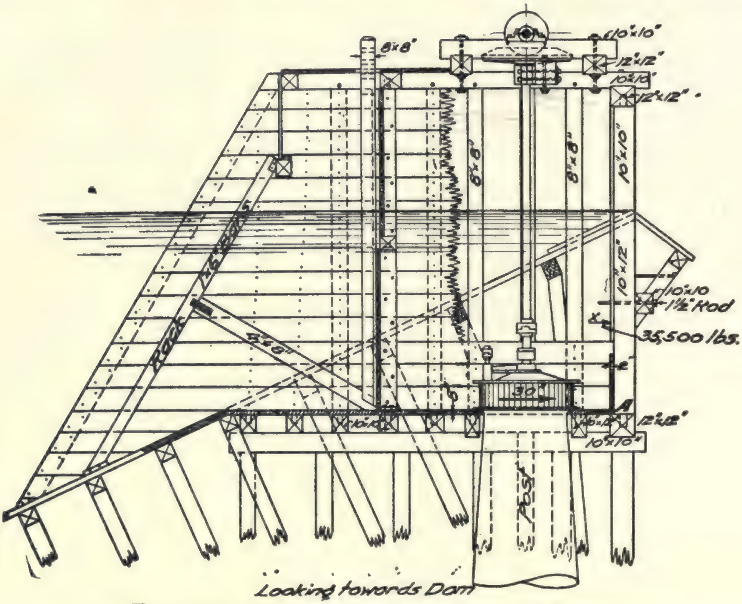


FIG. 293.—Timber wheel pits at end of dam.

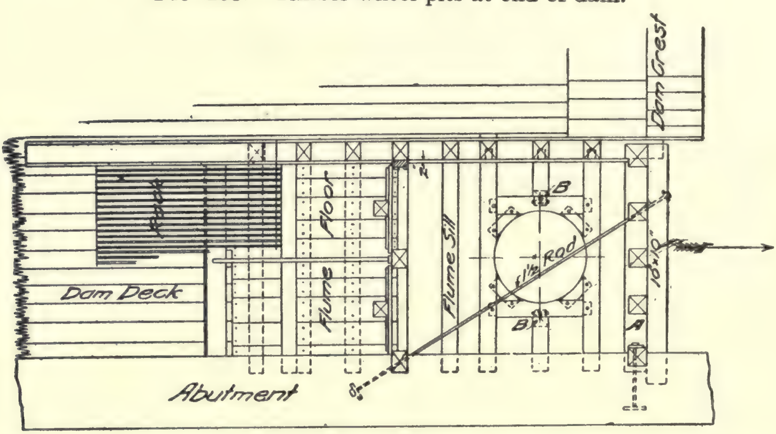


FIG. 294.—Timber wheel pits at end of dam.

them up and down stream. By building a masonry pier between the flume and the dam, any number of wheels could be set, as the masonry would resist the horizontal pressure.

Two posts are shown under the wheel setting at B. These are to support the wheel and are slanted off to each side of tail race. All planks are edged and seasoned. The head gate stems have holes two inches in diameter bored to permit the gates being lifted with a bar.

A light frame house should be erected over the gears. This same flume could, of course, be used for horizontal wheels. One plan is to have the shaft project through the down stream

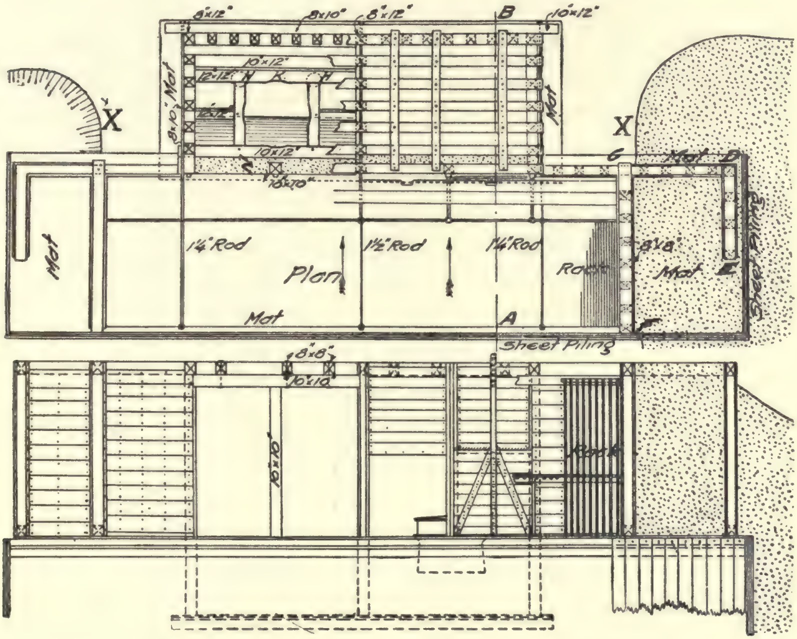


Fig. 295.—Timber wheel pits for soft bottoms.

bulkhead and have a rope drive to the machine back on the bank.

Fig. 295 shows two views of a timber flume built entirely separate from the dam and on a sand bottom, and Fig. 296 shows a section of the same flume. The wheel pit is first dug and the mat O laid. A concrete wall N is built as shown serving to resist the pressure of the sand under the upper mat and also as a deflection for the water discharged from the draft tubes. The timber frame serving to support

the turbines rests partly on this wall. The down stream sill, however, does not, and as it would have to sustain about 37,000 pounds of water, and a center load of about 5000 pounds, there must be placed at H, H, steel columns. Four-inch gas pipes with large cast iron bearing plates will serve this purpose.

As shown, there are settings for two turbines, but the same plan may be adapted to any number. Heavy rods are used to take up the larger part of the horizontal pressure. These rods are anchored to the up stream edge of the fore-bay mat by means of bolts put in before the sheet piling is driven. In this plan the fore-bay is widened to give liberal rack area. Sheet piling should be driven along both the up stream edge and the ends as shown. The plan view shows how the earth fill

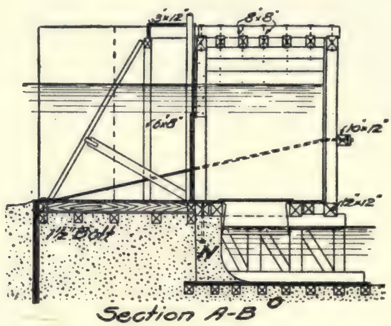


Fig. 296.

is only carried around to X. The wings $C\,D\,E\,F$ would be more lasting if made of concrete, which if reinforced and braced, need only be 12 inches thick. Where earth comes in contact with wet timber decay is very rapid. Frequently sound timber will, under such conditions, rot out in six years. The fore-bay mat must be made water-tight and the piling bolted and cemented to it. Two thicknesses of plank should be laid on the mat under the wheel pit.

The greatest difficulty is experienced in building a power house so that water will not follow around its sides, but the plan here shown should be safe from such accidents. All earth fills *must* be thoroughly tamped while *wet*.

Where there is no danger from back water, this plan may be used for horizontal wheels.

A compromise between the all timber and all concrete power house, is shown in plan and side elevation in Figs. 297 and 298. This is a very good plan for a power house of moderate cost. The cost could be further reduced by substituting timber for the concrete end walls shown at x. The only defect that developed in this plant was the splitting of the sill as shown. The use of a 1½-inch rod running through all three flumes would have remedied this weakness. Each flume contains one quarter turn 35-inch Morgan & Smith horizontal turbine.

This complete power house with a timber dam 22 feet high and 300 feet long cost $15,000.

Fig. 299 shows a late design of a concrete steel power house, The design of this power house embodies a novel and very important improvement. The waste gates, of which there are three for each, are used to draw down the head in time of flood. As shown here, the gates not only serve to pass large quantities of water, but they also act to increase the power of the turbines in times of high back water. This action is quite similar to that of a steam injector.

Fig. 300 shows a concrete steel power house of the most permanent kind. On account of being cramped for room the exciters and switch board are here placed on a platform above the generators. In reality the governors and generators are in the basement. It is seldom that the attendant has to tend the generators, the most of his time being spent with the switchboard and exciters, therefore this arrangement is permissible under the circumstances. The battlemented cornice shown on the building is made of building blocks but below this the building is built on the Thatcher plan. A feature of this plant is the method of emptying the reservoir. That part of the flume floor above the head gates is a continuation of the dam. At Y instead of the usual foot boards, boards are stood on end as shown, each board having a ring in the top by means of which it may be pulled out. The large window shown at end of generator room is to admit the generators, there being no other way in this case. The stairs are all steel and concrete. The floor and walls of the generator room are made water-tight up to the down stream windows to keep out the back water.

Fig. 301 shows the section of a power house which is designed

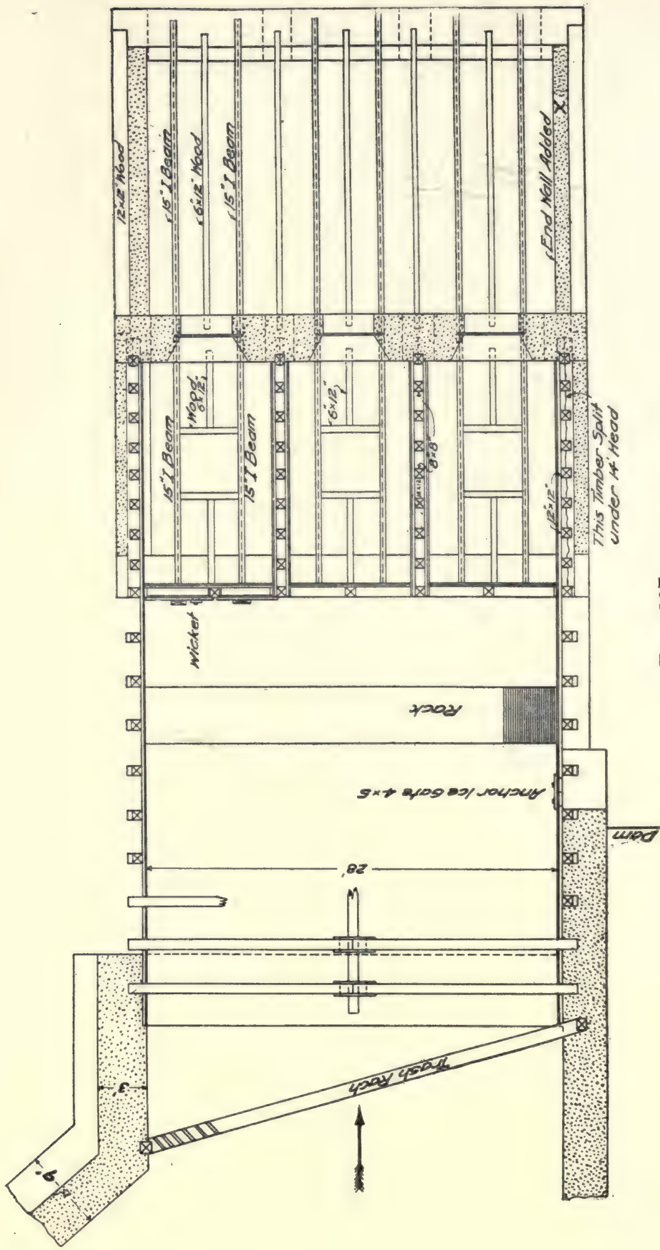

12"x12" Wood
15" I Beam
6"x12" Wood
15" I Beam
End Wall Added
Wood 6"x12"
15" I Beam
15" I Beam
6"x12"
8"x8"
12"x12"
This Timber Split under 14' Head
Wicket
Rack
Anchor Ice Gate 4 x 6
Dam
28'
Trash Rack
3'
6'

FIG. 297.

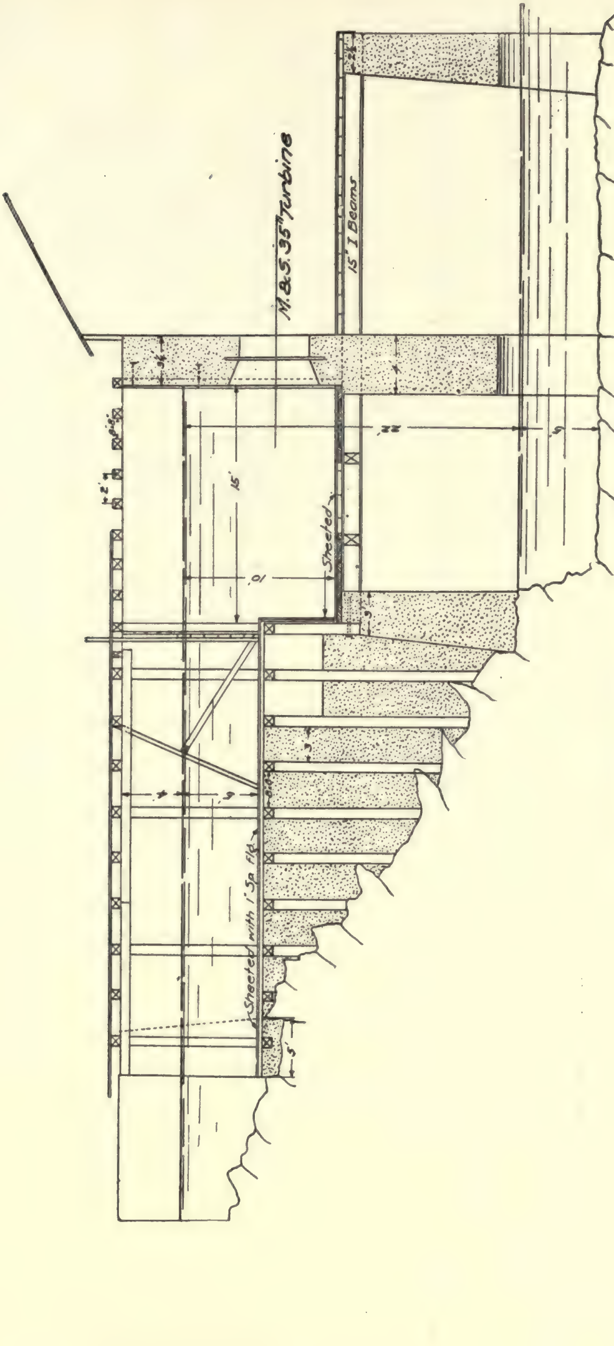
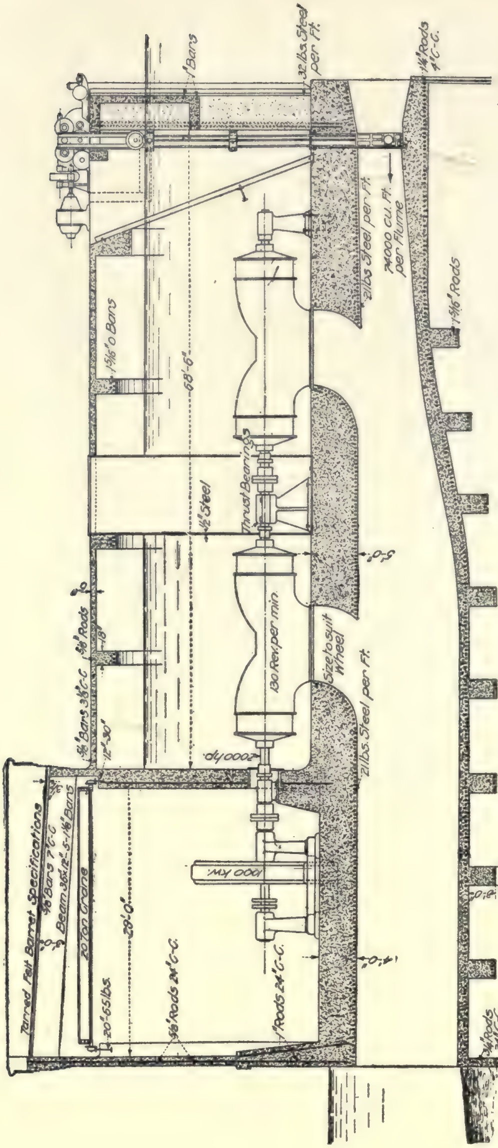
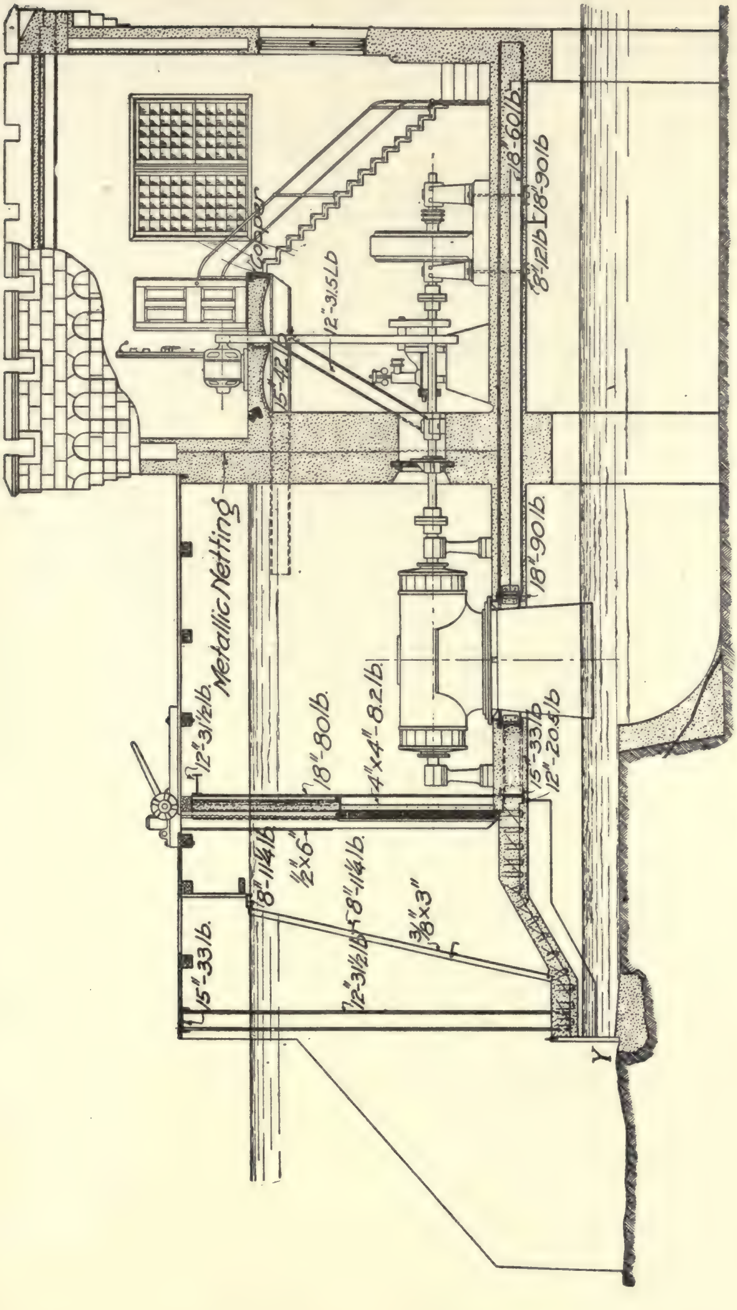


FIG. 298.—Power-house for medium heads.

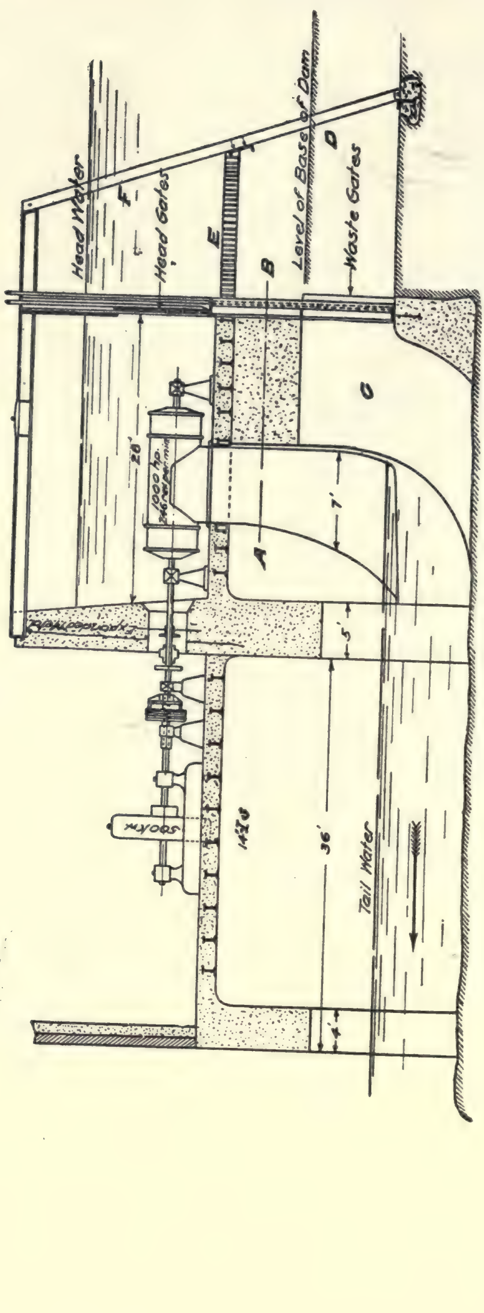


Designed for the Roberts & Abbott Company.

FIG. 299.—Modern power-house with ejector waste gates.



Designed for the Roberts & Abbott Company.
FIG. 300.—Reinforced concrete power-house.



Designed for the Roberts & Abbott Company.

FIG. 301.—Power-house for medium heads.

to waste the water above the dam through the wheel pits. Immediately above each draft tube a concrete wall *c* is built serving as a deflector and to protect the draft tube from the

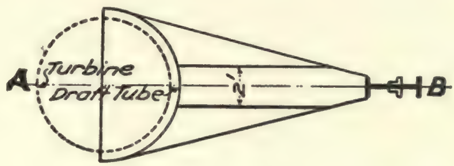


Fig. 302.

force of the water coming through the waste gates. Above the waste gates this wall is thinned to two feet, serving to support

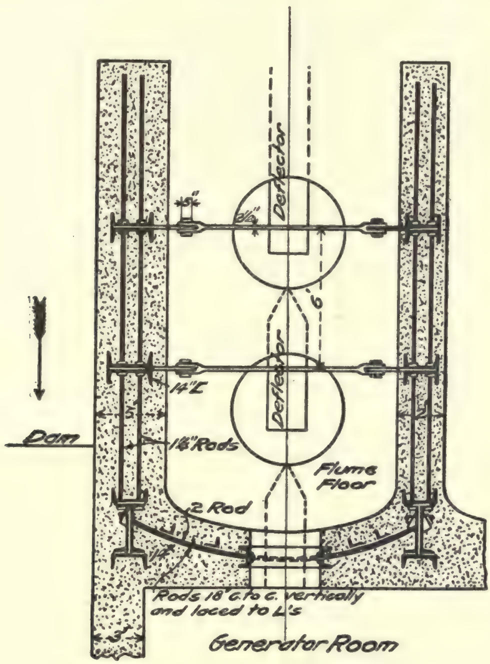


Fig. 303.—Excessive reinforcing.

the flume floor. At the up stream end the wall is only 12 inches thick.

Fig. 302 shows a view of the pier at *A B*.

The rack D is of heavy steel and serves to keep the trash out of the waste gates. At E is shown a steel rack which need not be used if the rack F has sufficient width.

Fig. 303 shows one of eight flumes, each containing two pairs of turbines. The head was only 20 feet though the heavy reinforcing would give the idea of a very high pressure.

This power house was built to cost as much as possible, and does not show good engineering. The arrangement of the deflectors is, however, very good. They serve also as piers under the flume floor. The part showing through the draft hole is curved to deflect the water while the up stream end is pointed so as to not choke the discharge from the turbines above.

The " Soo " plant is shown in Fig. 304. This is the largest low head power plant in the world. Each flume is built as shown in Fig. 305. Two sides are of I-beams filled in between with concrete and the end is of ¼-inch steel plates. This form of flume is patented, and the advantage claimed is that it takes up a minimum of space. It is a question in the author's mind how the expansion of the heavy steel beams will affect the adhesion of the concrete. Concrete reinforced with wire netting should be more efficient and less costly. ¼-inch steel is apt to rust out too quickly and sweats badly, causing a damp generator room. The space at A being of no value, would it not be a better plan to run the sides straight out and make the end square and of reinforced concrete? Or leave it round and make it of concrete reinforced with cables?

Fig. 306 shows in detail the setting of high pressure turbines. This plan is the latest in turbine installation for heads of 75 feet or more, and is the product of the Stillwell, Bierce & Smith Vail Company. All necessary relief valves and piping are here shown. The main valves are operated by water pressure.

When the head exceeds about 20 to 30 feet it is necessary to conduct the water to the wheels through penstocks. Figs. 307 and 308 show the power house of the Hannawa Falls Water Power Company. The head is about 85 feet.

The plan view shows an arrangement of penstocks made necessary by local topographical conditions and one not to be recommended. The head racks have an area of 200 square feet and since each 10 foot penstock carries 30,000 cubic feet per minute, the velocity through the racks, not allowing for the

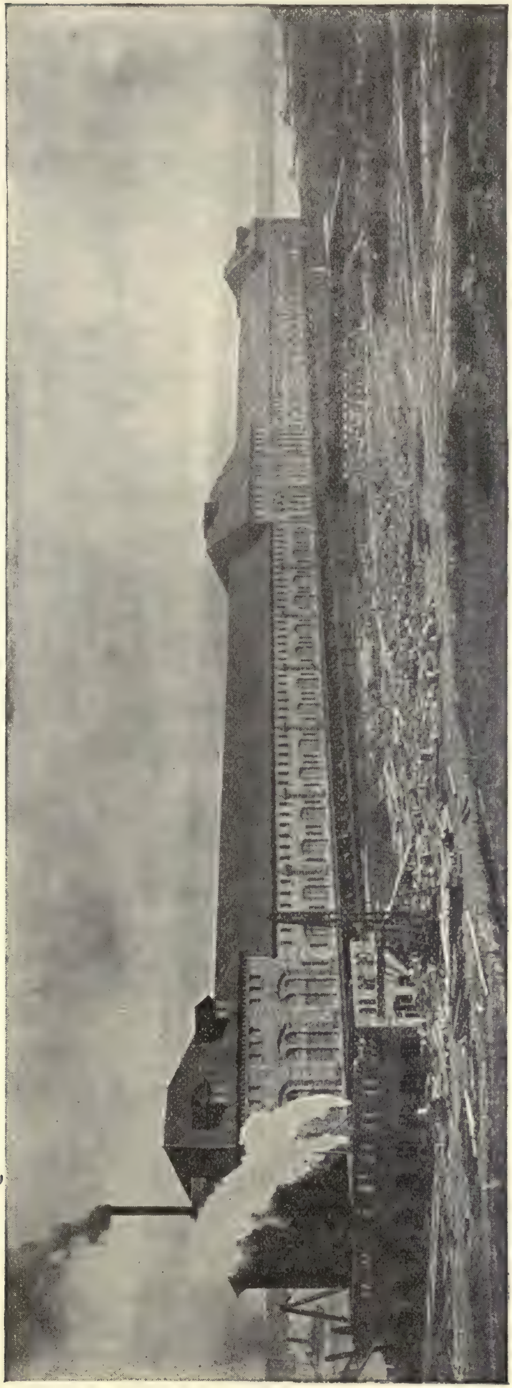


FIG. 304.—The "Soo" plant.

space taken up by the bars, would be 150 feet per minute, while it should be only 90. Another defect is the shallowness of the water under the draft tube, it being but five feet. The area of the tail race is about 500 square feet and the water discharged when all turbines are in use is 100,000 cubic feet per minute, giving a velocity of 200 feet per minute. As the percentage of head lost by this high velocity is small it may

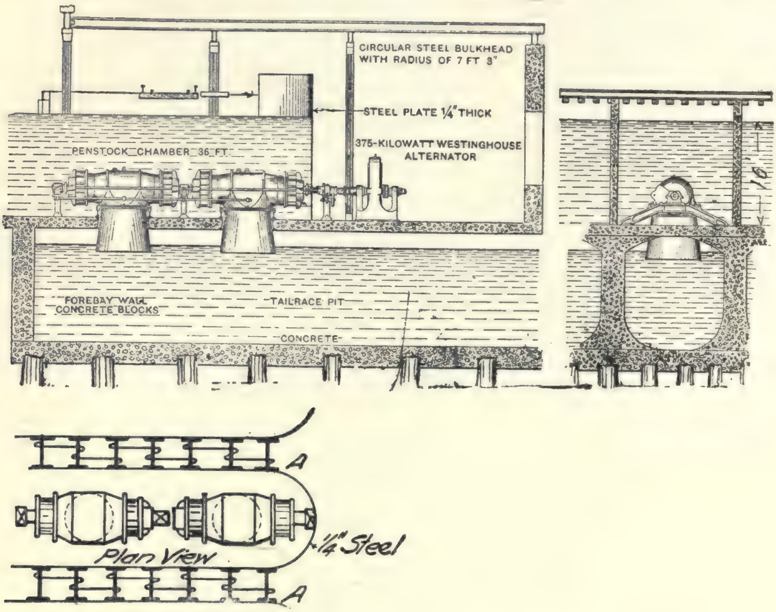


FIG. 305.—Details of the "Soo" plant.

be considered a good design. The penstocks are of 5/16-inch and $\frac{3}{8}$-inch mild steel, containing less than .06 per cent. of phosphorous. The turbines are special, having gun metal bronze runners of the Samson type. The gates are of cast steel, the case and draft-tubes of rolled steel and the remainder of cast iron. The upper part of the building is for manufacturing purposes.

Fig. 309 shows a well designed plant built for the largest paper mill in the world at Millinocket, Me. The part of the plant here shown is for the generation of 3000 kw. in three units. Three pairs of 36-inch turbines of 1500 h.p. each, drive three

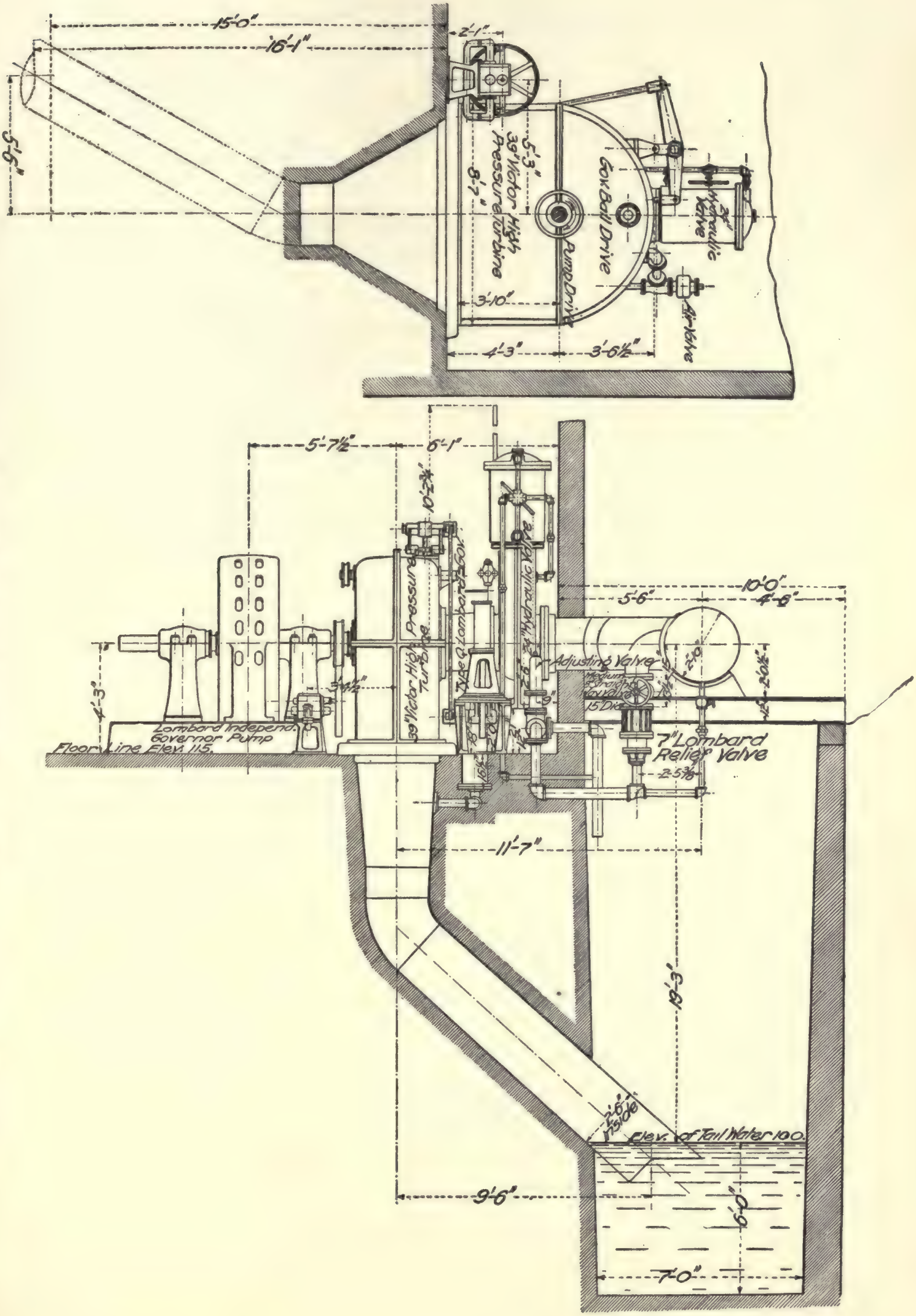
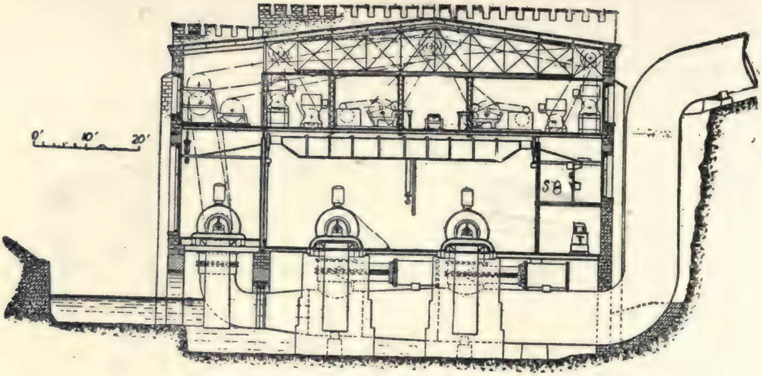


FIG. 306.—Turbine setting for high heads.



CROSS-SECTION OF THE POWER HOUSE.

FIG. 307.—Power-house.

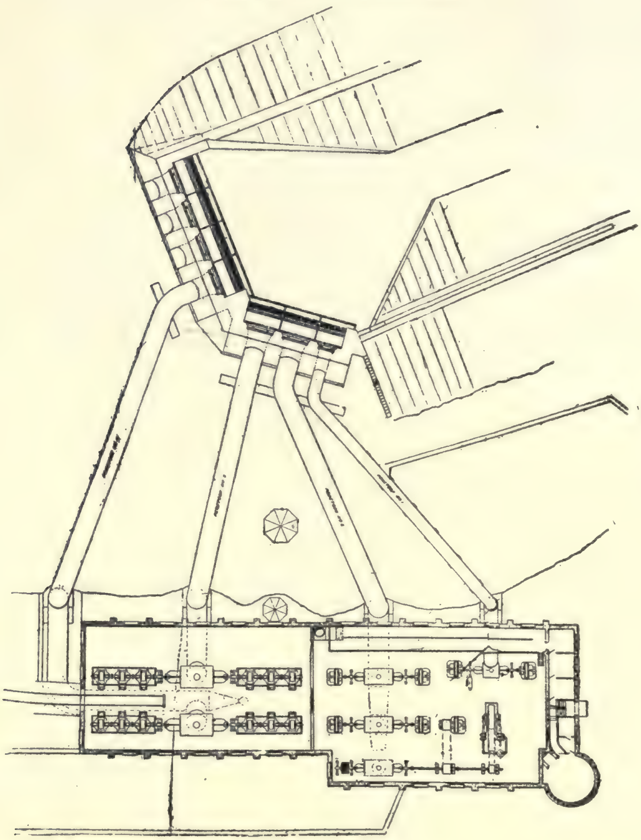


FIG. 308.—Power-house.

1000 kw. generators. Allowing for 80 per cent. efficiency in the turbines, 26,000 cubic feet of water are required per minute and for the exciters 1440, making 27,440 cubic feet per minute in all. This gives a velocity of five feet per second in the 11-foot penstock.

It will be noted that 1125 kw. turbines are used to drive 1000 kw. generators. This is evidently much too small as 10 per

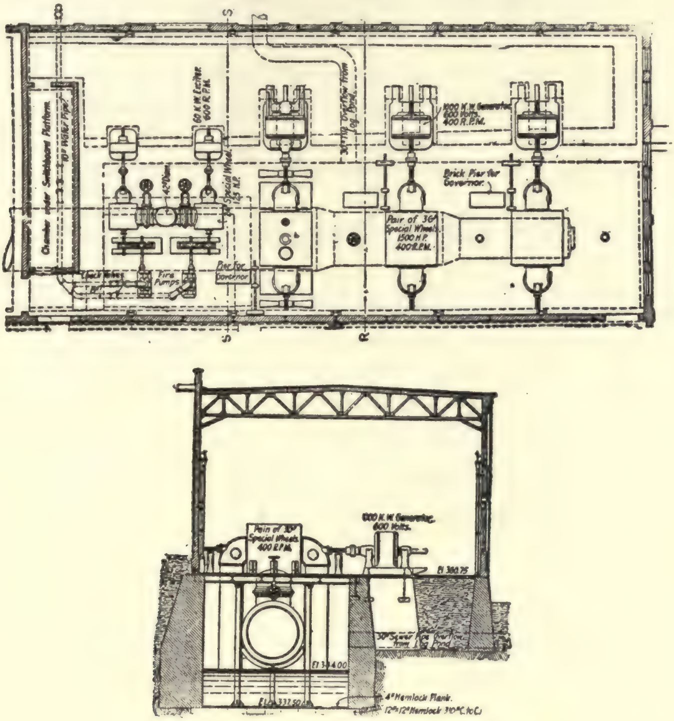


FIG. 309.—Power-house.

cent. of the turbine power is required for regulation and all generators should take a 50 per cent. overload for one half hour without overheating, therefore the full peak load capacity of these generators can never be used and fully 20 per cent. of the money invested in them is bringing no return. Each pair of turbines should be of 1400 kw. capacity, because most turbines are most efficient on $\frac{7}{8}$ gate, the turbines depreciate in efficiency

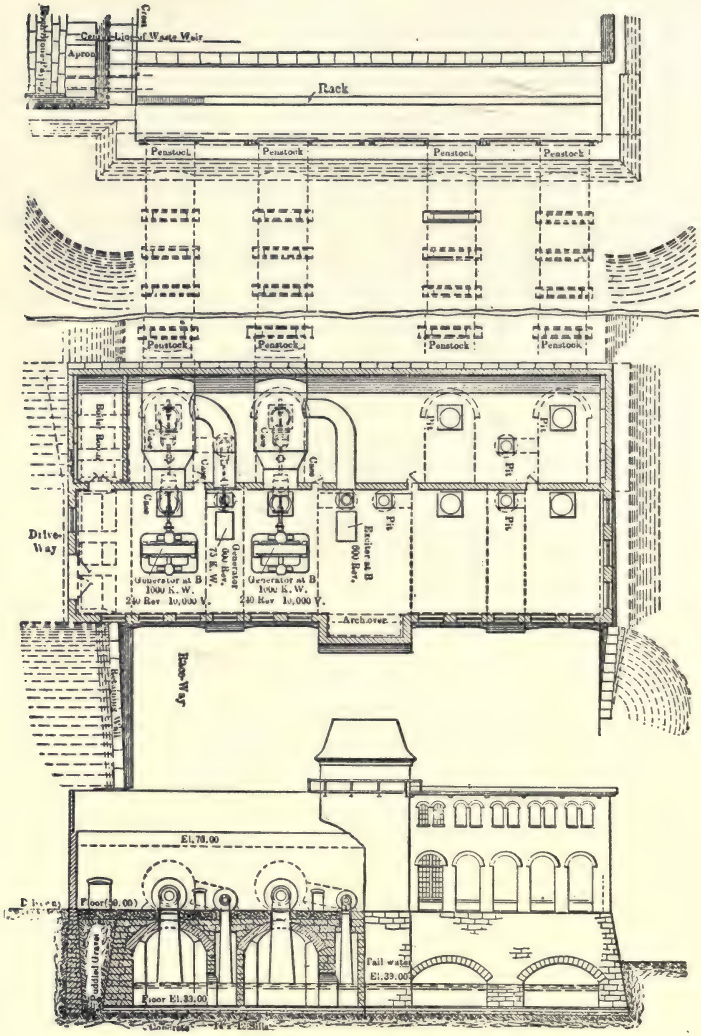


Fig. 310.—Power-house.

much more rapidly than do the generators with the result that in five or ten years the turbine power will become too weak for the maintenance of the proper voltage in the generators.

An interesting feature of this plant is the manner of bringing in the penstock *under* the turbine.

Fig. 310 shows common type of power house for medium heads. In this case the head is about 40 feet. The steel penstocks are

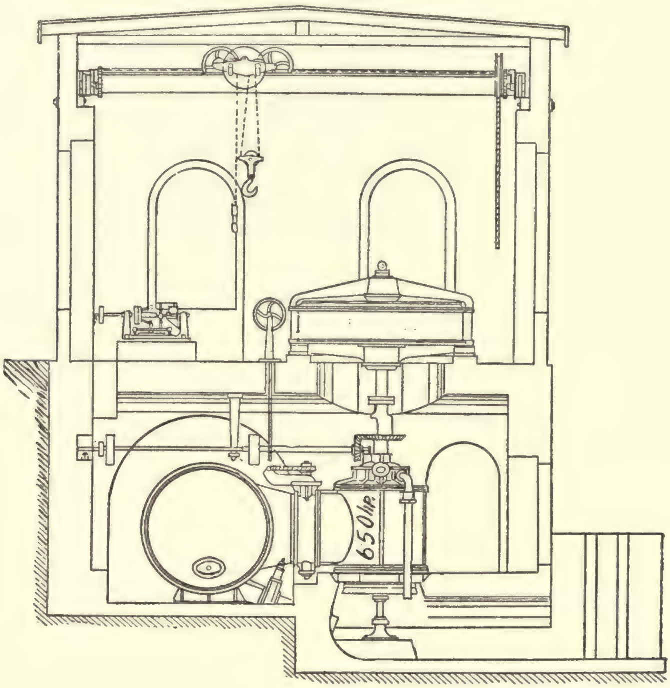


FIG. 311.—Foreign design for hydroelectric power-house.

brought into the power house through a masonry wall. A second wall separates the turbine cases from the generator room, thus insuring a dry generator room. The power house was on a soft bottom and rests on a heavy timber mat, no piling being driven to sustain it. This plant is at Red Bridge, Mass.

To give the reader some idea of foreign practice a typical plant is shown in Figs. 311 and 312. In Europe vertical direct

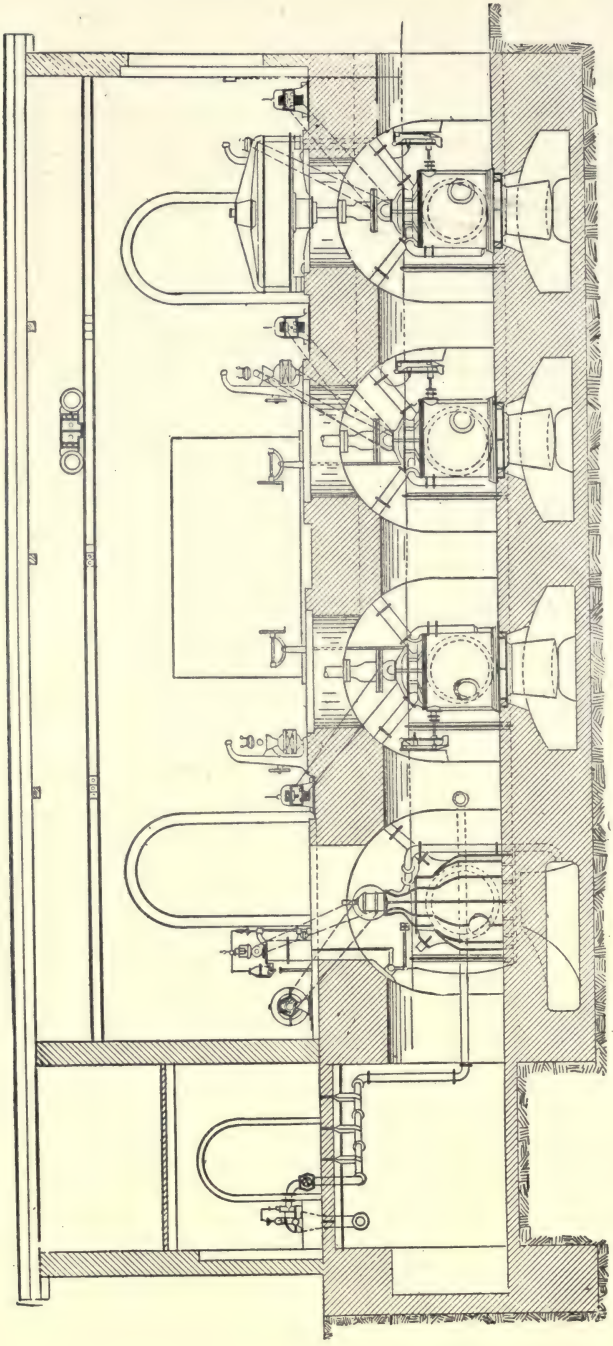


FIG. 312.—LONGITUDINAL SECTION OF LA GOULE GENERATING STATION.

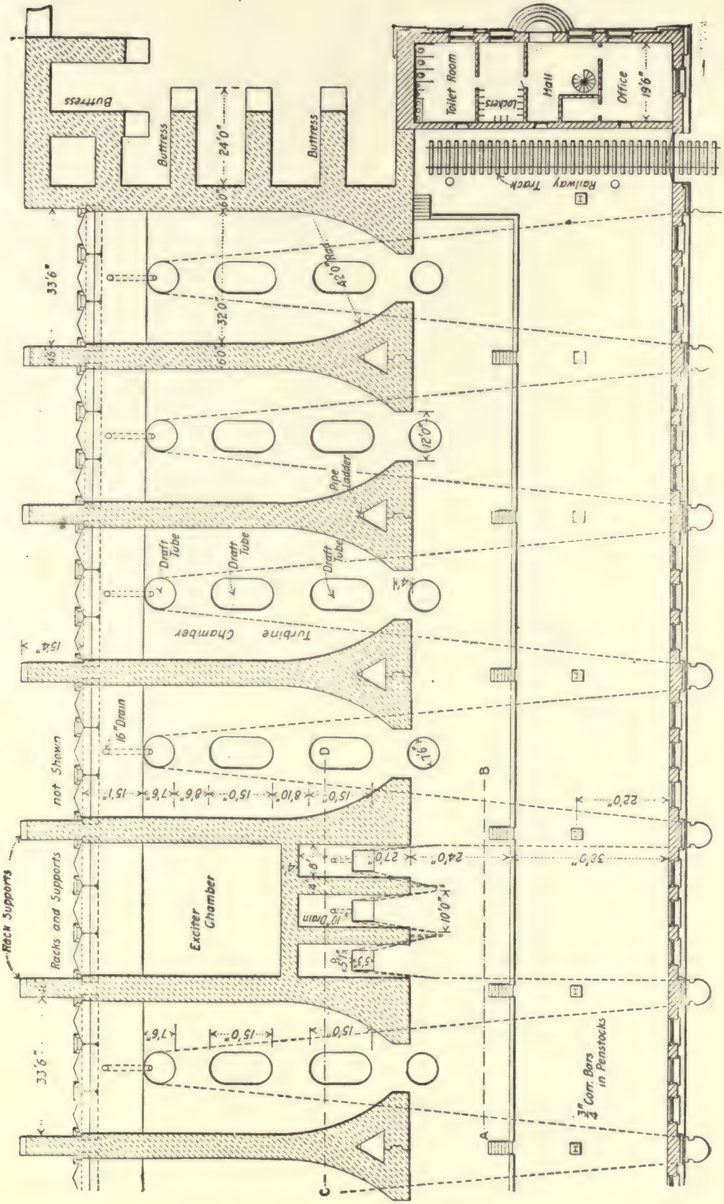


FIG. 313.—Plan of Power-house on Chicago Drainage Canal.

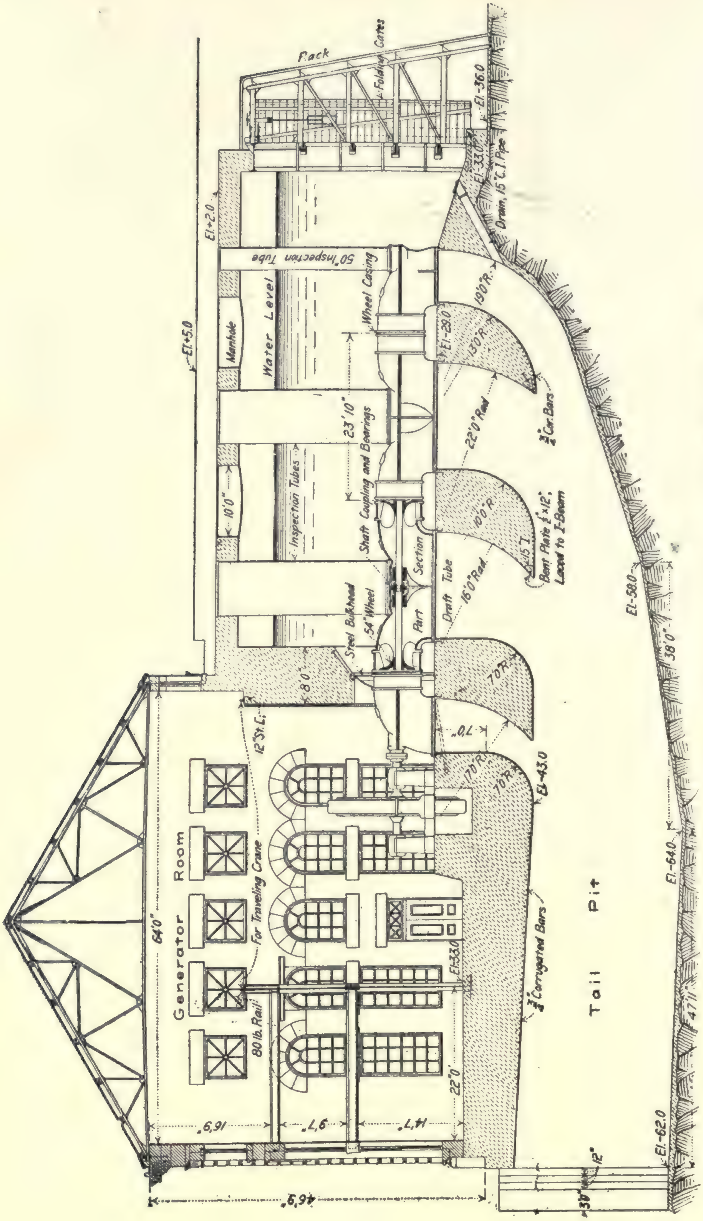


FIG. 314.—Section of Power-house on Chicago Drainage Canal.

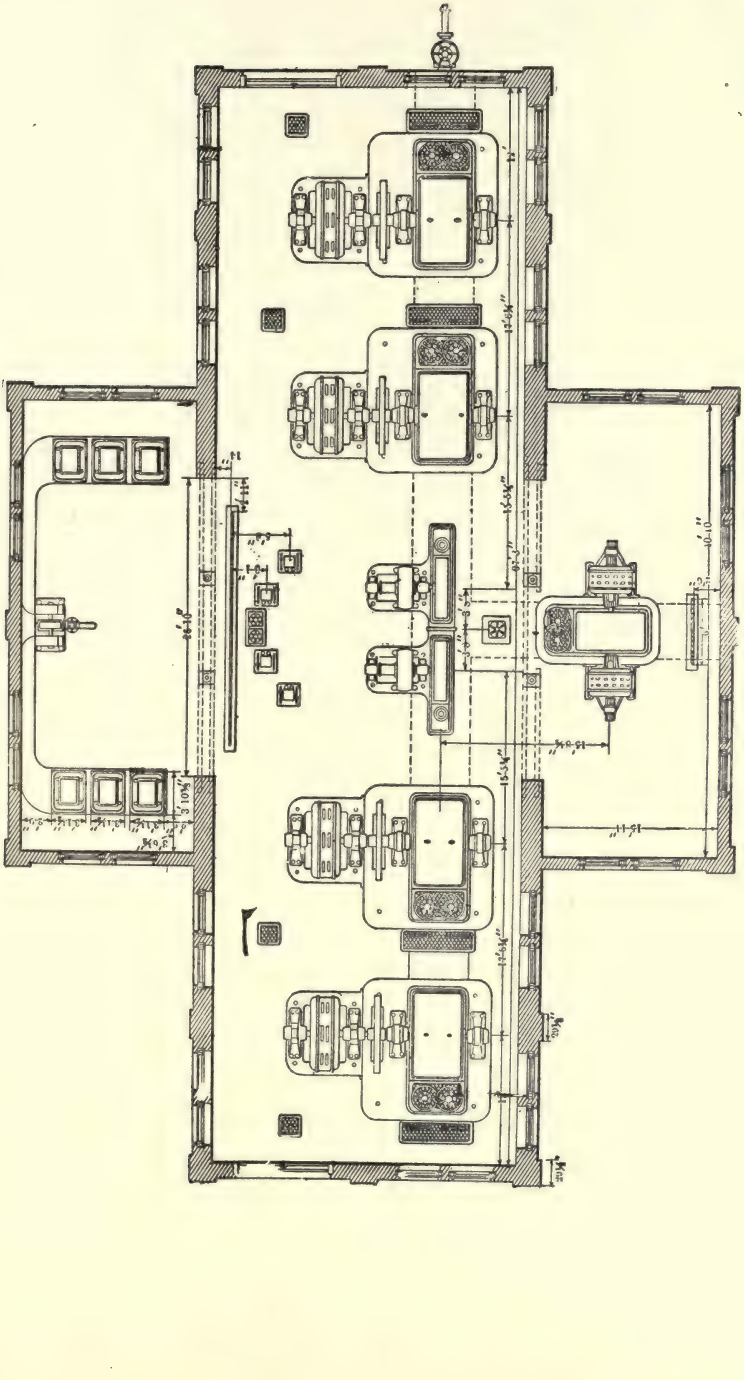


FIG. 315.—Plan of Power-house for high heads.

connected generators are quite common. This plant works under an 85-foot head and develops over 2000 h.p. Three of the generators are 500 h.p. driven by turbines of 650 h.p. In this case partial allowance has been made for regulation and the peak load of the generator so that the full efficiency of the generators can be obtained. Under test the turbines gave 81 per cent. efficiency, at $\frac{3}{4}$ load and 79 per cent. at full load. Therefore, when developing 487 h.p. the turbines are the most efficient. When the highest efficiency is desired, even in this case, the turbine capacity is too small, as at $\frac{3}{4}$ load it should drive the generator under a 50 per cent. overload and the

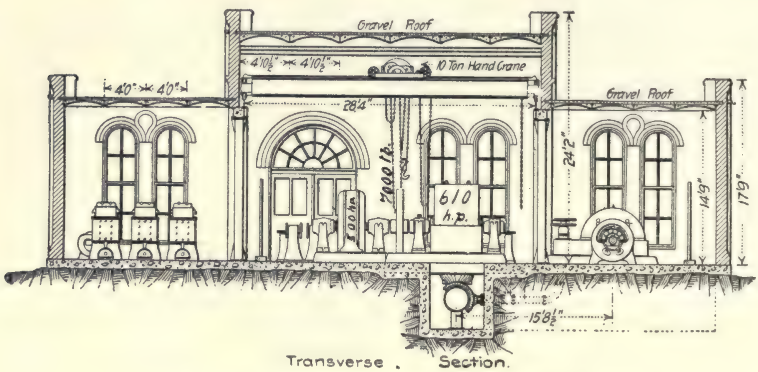


Fig. 316.—Power-house for high heads.

governors. However, the above is better than the average American practice. Governors made by Escher, Wyse & Co. (Allis-Chalmers Co. are the American manufacturers) are used, these being a standard make in Europe, and are becoming better known in America.

Figs. 313 and 314 illustrate the power plant now being built for the utilization of the waters of the Chicago drainage canal. It would be difficult, indeed, to criticize the design of this plant, except that more reinforcing might have been used in the concrete, but on the whole this plant marks a distinct advance in such construction.

Figs. 315 to 317 give a good idea of a pelton water wheel plant built for the Pike's Peak Power Company. The effective head is 1160 feet. Each of the four peltons driving the gen-

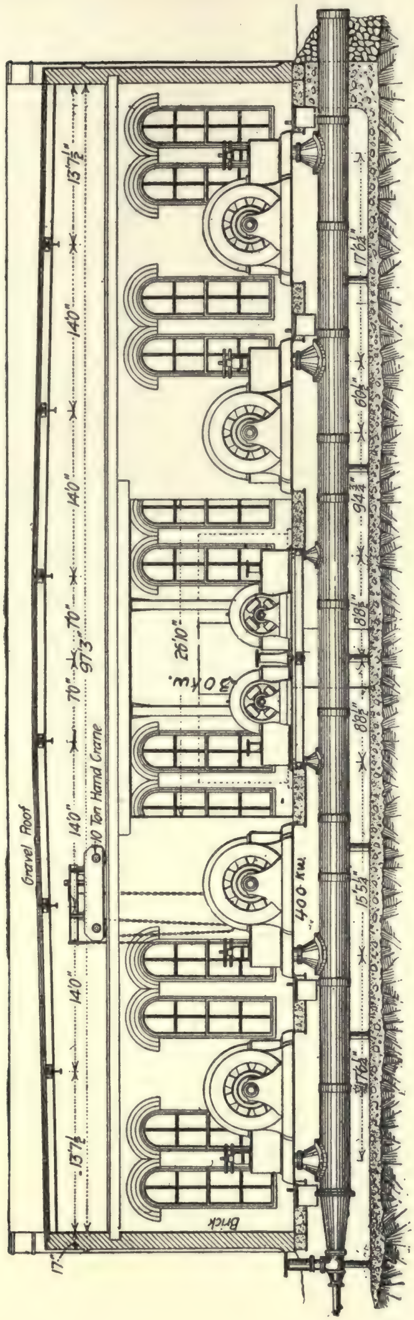


FIG. 317.—Power-house for high heads.

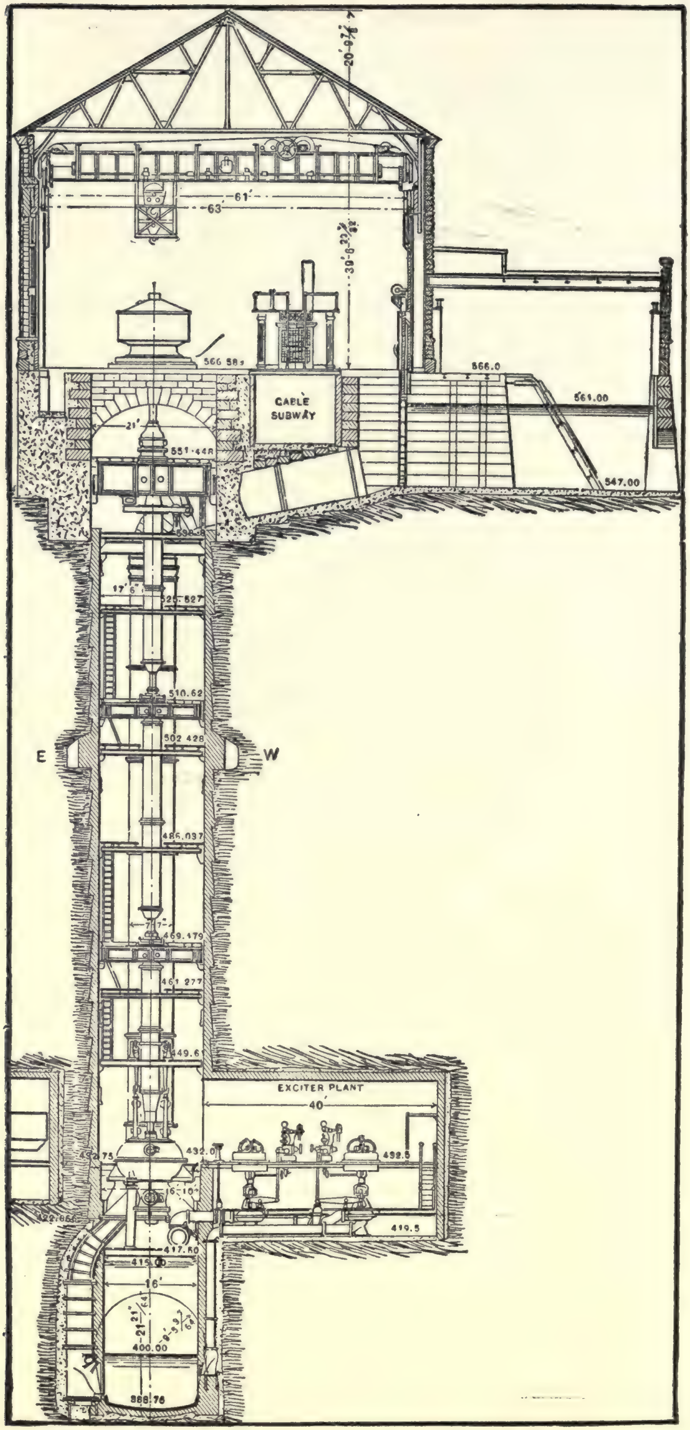


FIG. 318.—Power-house at Niagara Falls.

erators is of 610 h.p., though by using one of the two wheels comprising each unit and by using different sized nozzles, almost any power can be obtained at full efficiency. Each pelton unit drives a 500 h.p. generator at 450 r.p.m. This plant was tested and the efficiency from water to switchboard was 78 per cent. All piping was tested to 800 pounds pressure per square inch.

Fig. 318 shows a 5000 h.p. unit in the most noted hydro-electric plant in the world, namely, the Niagara plant. The turbines were designed by Escher, Wyse & Company of Zurich,

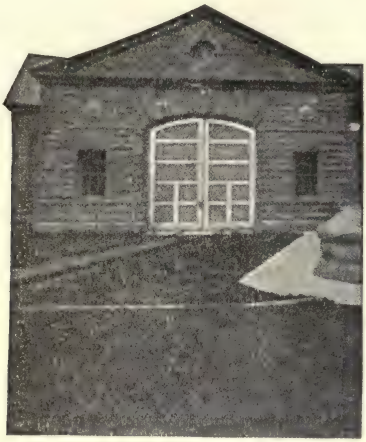


FIG. 319.—Example of power-house architecture.

Switzerland, and built by I. P. Morris Co., of Philadelphia.

The governors were the design of the same Swiss company, and were built by A. Falkinan of Philadelphia. These governors permit a speed variation of five per cent. from full to no load, and for ordinary load variation is as good as modern steam engine practice. The generators are of the two-phase type. Much of the power is used by electric furnaces using single phase current. This unbalances the generator load and taxes the regulation to the utmost. The regulation of voltage is within ten per cent.; the efficiency of the generators is 98 per cent.; the working head on the turbines is 161 feet.

ARCHITECTURE.

It costs but very little more to give to the exterior of a power house or head works a fine appearance, and in after years the appearance may be an important factor in the price for which the property will sell.

FIG. 320.—Architecture suitable for head-gates, arches, etc.

Figs. 319 and 320 show some types of construction which have been used on numerous well-know structures, and may aid the engineer in the design of hydroelectric power plants.

CHAPTER VII.

POWER HOUSE EQUIPMENT.

WATERWHEELS.

TURBINES.

Like all other matters pertaining to hydraulics, the turbine has made slight progress in the last 50 years. In 1840 the Swain turbine gave 80 per cent. efficiency on test and the Leffel 74 per cent. The following is a list of some of the most prominent turbines. The efficiency given is the catalogue value, and even an efficiency of 80 per cent. is seldom guaranteed. The only improvement has been in size, and speed and efficiency at part gate.

TABLE XXXIX.

COMPARISON OF VARIOUS MAKES OF WHEELS.

Wheels.	Diameter.	Head.	Revolutions.	H.P.	Water cu. ft. per mm.	Efficiency.
Hercules	30 inches	20 feet	174	119.59	3,960	80%
Samson	30 "	20 "	242	162.00	5,312	80%
McCormick	30 "	20 "	186	142.70	4,721	80%
Victor	30 "	20 "	210	165.35	5,471	80%
Rice's Victor	30 "	20 "	222	183.72	6,079	80%
Hunt	30 "	20 "	187	111.52	3,556	80%
Swain	30 "	20 "	197	100.78	3,273	82%

Most of the turbine makers give the results of tests performed at testing flumes, proving high efficiency, but it is the author's opinion that little dependence should be placed on these. There is no question but that the tests are correctly performed but the wheel makers do not give to the public all the data connected with the test. This is known to have been the case in several

instances. The only safe way is to have a written guarantee from the makers.

Turbines may be divided into three general classes which will serve the purposes of this book: Register gate, wicket, and cylinder gate. All turbines are now made in both the horizontal and vertical forms. They all have a runner of the same general type. Fig. 321 shows a Victor runner and it typifies many others. This is the part of the turbine which revolves. The buckets are usually of steel cast into the cast iron frame.

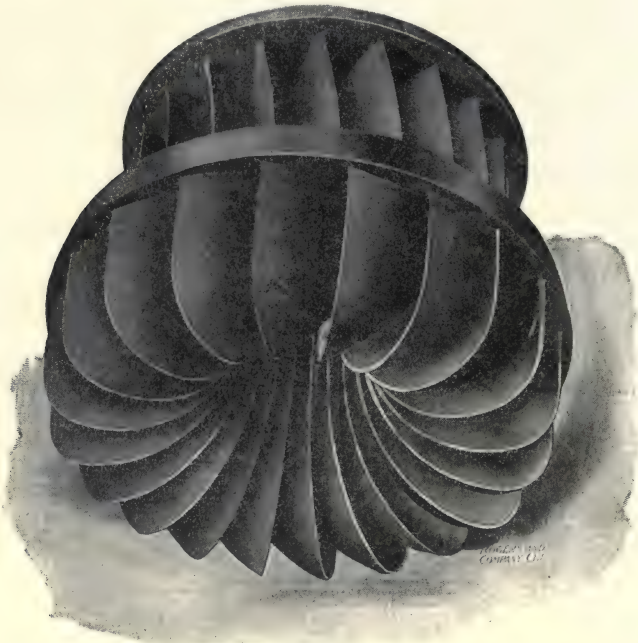


Fig. 321.

Among the wicket gate turbines the Leffel and American are the most prominent. The former has a greater speed than any other, and in construction is one of the strongest.

In the latest Leffel and Victor wheels the gates are operated by a ring and lever (Figs. 360 and 324) instead of the numerous rods shown in Figs. 322–323. This is an important improvement where a sensitive governor is used as the number and weight of the moving parts are greatly reduced. This wheel

can be used for heads up to 40 feet. They have a good part load efficiency.

Turbines built by the S. Morgan Smith Company are for the most part of the cylinder gate type. A form of wicket gate as made by the above-mentioned company, is shown in Fig. 324. It's mode of operation is immediately apparent from the illustration. Fig. 325 shows a double turbine built by the S.

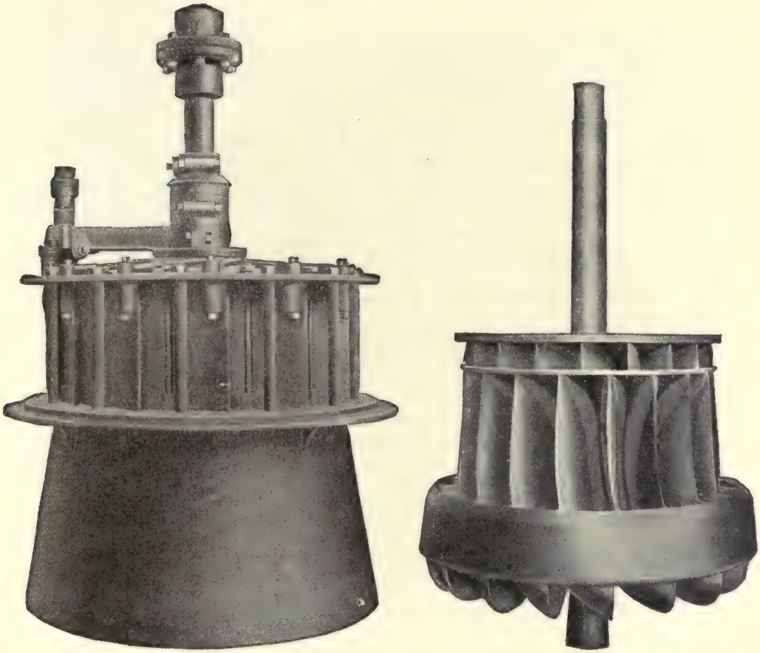


FIG. 322.

Morgan Smith Company. This turbine, as will be seen from the controlling devices, is of the cylinder gate type. It was built to operate under a head of 85 feet.

The greater proportion of turbines are of the cylinder type. Fig. 326 shows a sectional view of a Victor turbine made by the Platt Iron Works Company of Dayton, Ohio. This is their latest wheel and the invention of A. C. Rice; A and A' are the runners, and F and F' the cylinder gates operated in the direction of shaft by the rods a and a'. The gears operating these gate rods are run in oil and project through the bulkhead

into the power house. All cylinder gate turbines have a gate similar to the Victor gate. The cylinder gate is more nearly water tight than the registering gates, but taken with the counter weight used to balance them they are heavier than the register.

The Platt Iron Works Company makes a high pressure turbine which can be used with heads of from 70 to 700 feet. They thus fill in the gap between the ordinary turbine and the

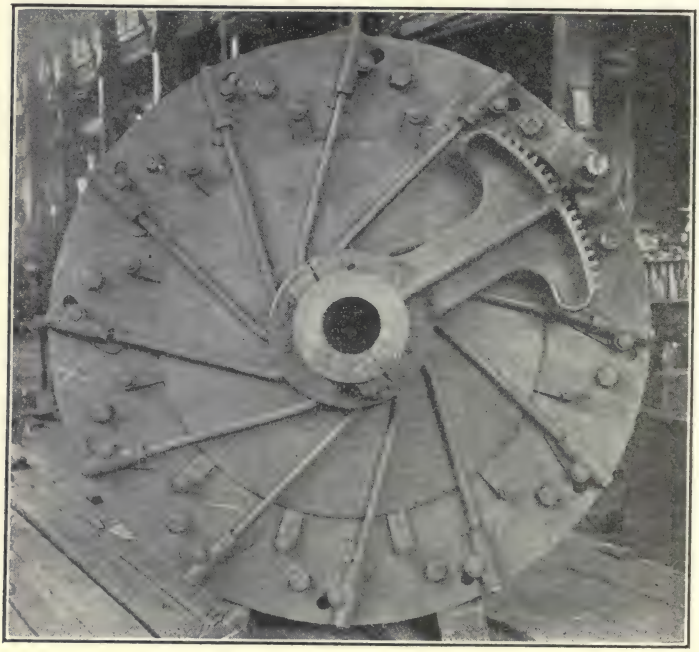


FIG. 323.

Pelton. In operating turbines under high heads it is imperative that all grit be removed from the water before it is passed through the wheels, otherwise the wheels soon wear out. On such wheels all running parts should be made of bronze. The action of water under high pressure is such that holes are often bored through solid cast iron by the impact of the fluid.

Under heads above 50 feet, an efficiency of 75 per cent. is very good, and for the average gate opening this is too high, for heads above 200 feet 80 per cent, is a fair efficiency.

Wheels tested at Holyoke, Mass., are tested under the most favorable conditions. The wheel is new and smoothed up unusually well. The setting is perfect, and all parts are new and water-tight.

After a year or so of use all turbines become more or less leaky, out of alignment and the buckets become dented and rough. It is no uncommon thing for the $\frac{1}{2}$-inch steel buckets to get knocked partly out of the casting.

There are numerous forms of turbine settings, a few of which

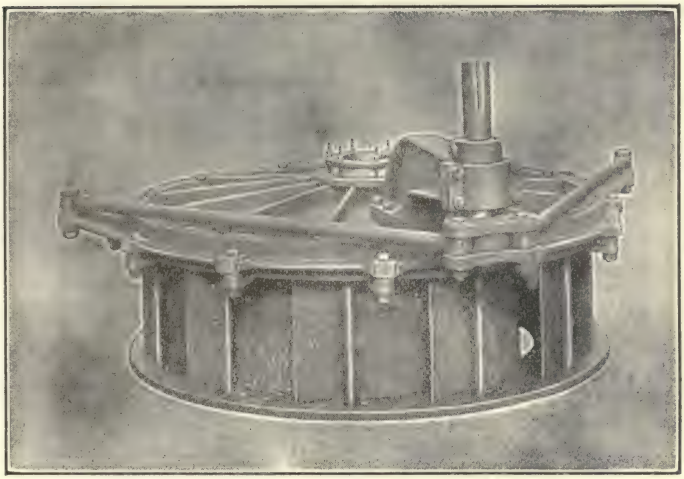


FIG. 324.

are given in Figs. 327 to 341. Cf course any slant may be given the various pipes and draft tubes.

There has been such a fad for horizontal turbines that they were often installed where the vertical type would be preferable. The horizontal turbine allows the placing of two or more turbines on one shaft, thus getting increased speed with the same power or increased power with the same speed. However, their use usually necessitates a generator of slower speed than would the vertical type, and hence of more cost and makes regulation within wide limits of head an impossibility. When

the frequency is not of prime importance, the ordinary excitation of a generator can take care of a 10 per cent. reduction in speed, but beyond this the voltage will fall, therefore, where

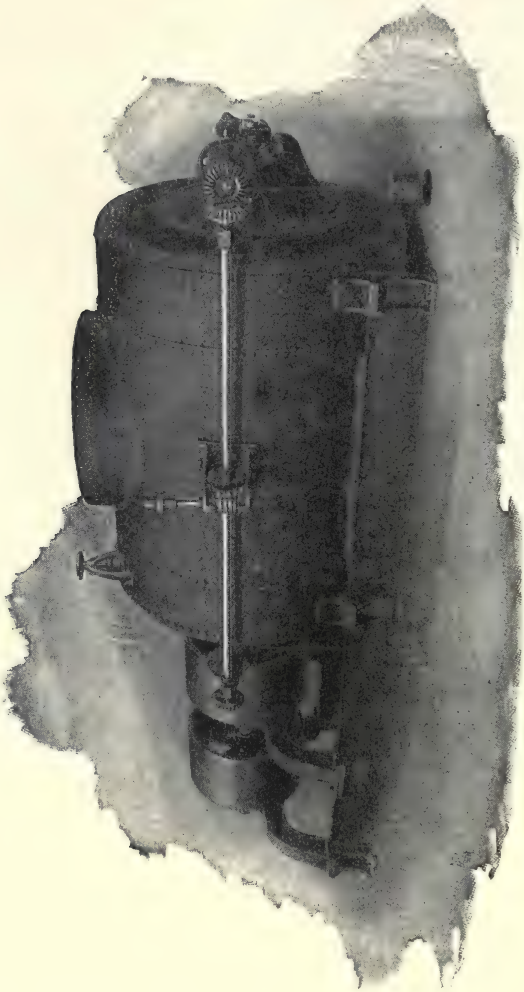


FIG. 325.—Pair of cylinder gate turbines. Note the counterweight.

severe reduction in head is to be apprehended, the vertical turbine with its noisy gearing may be the best practice as with

a proper ratio of gearing and a sufficient number of turbines the speed of the line shaft can be kept up.

The determining factors as to the selection of the horizontal turbine should be: One, the speed variation under all probable stages of head and back water; two, additional cost of machinery, due allowance being made for the increased efficiency

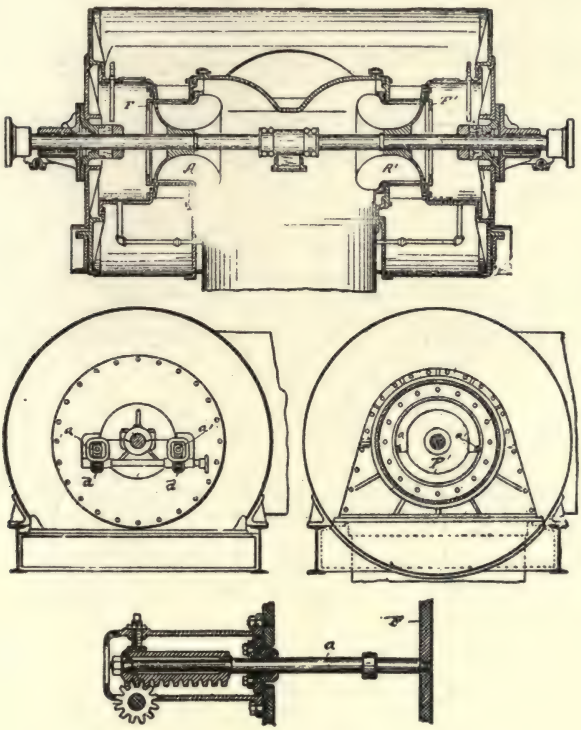
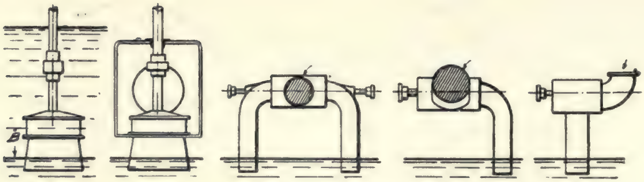
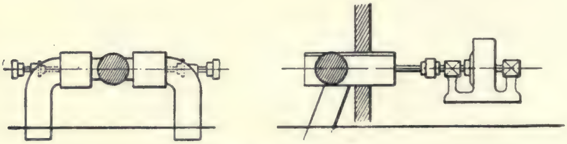


FIG. 326.

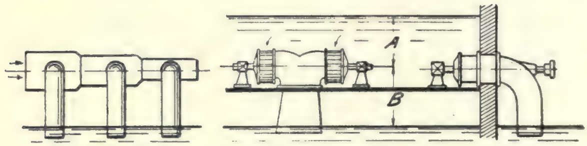
of the horizontal wheel. (The same wheel mounted in a horizontal position would have an efficiency about three per cent. less than in the vertical position, but since about 10 per cent, or more is lost in the gearing, etc., the turbine will be about seven per cent. more efficient in the horizontal position than in the vertical;) three, cost of turbine setting.



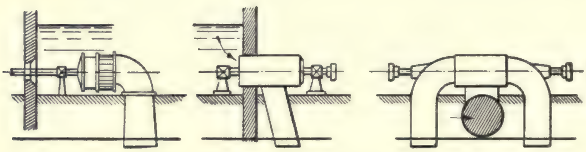
FIGS. 327-331.



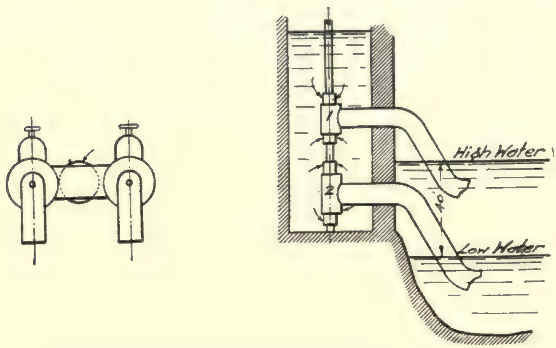
FIGS. 332-333.



FIGS. 334-336.



FIGS. 337-339.



FIGS. 340-341.—Typical Turbine Settings

There must be at all times, from six feet to ten feet of water at A, Fig. 335, to prevent air bubbles and whirlpools. Horizontal wheels have been used on heads as low as 15 feet. The great Soo plant has 16 feet head and the turbines (Hunt) gave 84 per cent. efficiency on test

It sometimes happens that a water power is to be developed at a head considerably below that to which it will be increased later on, and it is desired to install generators and turbines which will answer for both heads. The following setting was designed by Mr. M. E. Powers (Fig. 342). The gears were so

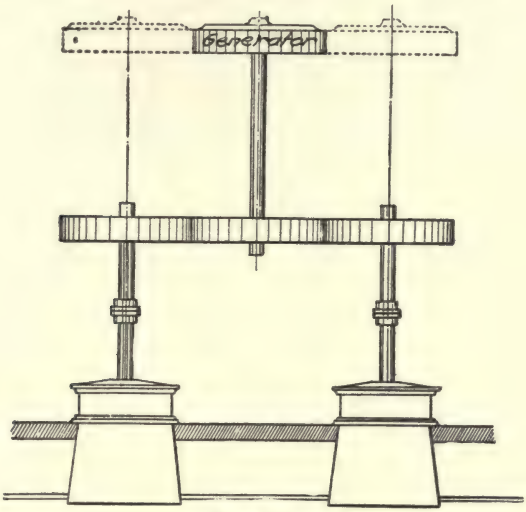


FIG. 342.

proportioned that at the reduced head the generator had the proper speed. The turbines were selected so that under the increased head, the generator shown in sketch could be moved directly over it and direct connected to it. A second generator was then installed and set over the other turbine. The gears were therefore all there was to be discarded.

Wherever possible it is best to place two, four or six turbines on a shaft so as to balance the end thrust (see Figs. 329, 332, 335 and 339).

An open setting, Figs. 327, 335, 336 and 337, should be obtained when possible, the efficiency being greater and the governing better.

Fig. 341 shows about the only alternative where the variation between high and low water exceeds the maximum practicable length of draft tube, and where it is desired to maintain the speed and power with direct connected units. The back water wheel No. 1, runs idle during normal water, but is used when the back water reaches the draft tube.

Next to lack of water during the months of minimum flow, the reduction of head by back water is the most serious osbtacle the hydraulic engineer has to contend with. One good feature, however, is that we have plenty of water, so that by installing enough turbines and properly gearing them, we may keep up the speed of the line shaft and also the power.

Ordinarily building the dam does not affect the stage of back water. Its cause lies below the dam, and is due to the choking effect of the river banks, islands, bends, etc. Usually the high

FIG. 343.

water mark can be distinguished by the driftwood along the banks. It is seldom indeed that there are not somewhere sure indications of the high water mark. Farmers along the river can corroborate the evidence, so that there should be no trouble in determining the back water stage.

The depth of water over the dam is, as a rule, more difficult to pre-determine. If there is a dam anywhere above, the problem is an easy one. As a rule, if you build a dam in a river having parallel shores, as in Fig. 343, the water will pile up below the dam twice as much as it does above. If the dam is narrower than the average width of the stream, the difference will of course be less.

Take an example where we have 20 feet of head at normal stages. Suppose we frequently have the head diminished five feet due to back water, and in extreme cases our head is reduced to ten feet. Then for a series of turbines all having the same power with same head, we have the following:

EXAMPLE:—We wish to drive a 350 h.p. generator at 290 r.p.m. and we do not want the speed to fall below 260 as the field rheostats of the generator will not take care of a greater variation. The *normal* flow of the stream will just supply the one wheel.

Referring to tables for the Samson turbine we find that a 50-inch Samson will give 451 h.p., at 145 r.p.m. under a 20-foot head.

First. Under *normal* conditions the first pair of gears (Fig. 344) will be in the ratio of 2 to 1 and turbine No. 2 will be thrown out of gear.

Second. The head becomes reduced to 15 feet and the speed of the generator falls to 252 r.p.m. Now if but one turbine were in gear the ratio of the gears would be 1 to 2.3 to keep the speed at 290, as under 15-foot head a 50-inch Samson has 126 r.p.m.;

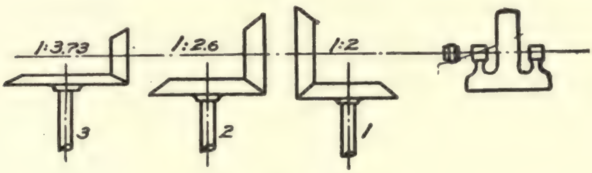


FIG. 344.

but the power would be too low, being only 293 h.p. We therefore gear turbine No. 2 in the ratio of 1 to 2.6. The speed of the line shaft will then be a mean between the two sets of gears.

$\frac{2+2.6}{2} = 2.3$ and $2.3 \times 126 = 298.8$. With the two turbines we have under 15-foot head 586 h.p. We have therefore kept up the speed and power.

Third. Head reduced to 10 feet; three 50-inch Samsons under 10-foot head will give 480 h.p. at 103 r.p.m.

With turbines No. 1 and 2 geared as above the speed will be 237 r.p.m. Now if we add a third turbine we do not double the power as before, but have to allow for the action of a one-third power acting on the two-third power.

We have the following formulas for any number of turbines:

$$X = \left(\frac{R\ N}{R'}\right) - (A + B +)$$

where X = the ratio to be found; R = the number of revolutions desired on the line shaft. N = the number of turbines including the turbine the ratio of whose gears it is desired to find; R' = the revolutions of the turbines under the reduced head (found from turbine tables) at which the n'th turbine is thrown into gear. A = the ratio of the gears of turbine No. 1. B = ratio of gears of turbine No. 2. C would = ratio of No. 3, etc.

Thus in the above example, $X = \left(\frac{290 \times 3}{103}\right) - (2+2.6)$; $X =$ 3.846. If we add another turbine for a head of only eight feet we find that all the turbines will have a velocity of 92 r.p.m. and $X = \frac{290 \times 4}{92} - (2+2.6+3.73) = 4.27$ as the ratio of the fourth set of gears. The power is 456 h.p.

Care must be taken in keeping the proper proportions for the gears; that is, the speed of the teeth, pressure, etc.

It is this ability to regulate the speed and keep up the power that often makes it desirable to install vertical rather than horizontal turbines. Of course, the head works must be given sufficient area to take care of the great quantity of water used under the reduced head.

During periods of flood, a velocity of 100 feet may be used through the racks and 200 feet through the tail race as efficiency is not so important at high water as are speed and power.

Since every turbine has a certain velocity at which it is most efficient the above arrangement will not be of high efficiency, but at the time the extra turbines are thrown into gear there is plenty of water. The lower geared turbines are made to race while the higher gears cause the turbines to work at a low speed. Under extreme conditions of back water the low geared wheels may develop no power at all in which case they would be cut out.

By the use of the draft tube any turbine may be placed above tail water. This distance B, Figs. 327 and 336, is theoretically 34 feet, but in practice there are reasons why the length should be about as given in the table by Mr. John Wolf Thurso.

Diameter of draft tube in feet.	= 0.5	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Draft head in feet.	= 32.5	30	27.5	25	22.5	20	18	16	14.5	13	12	11	10.5	10	9.5

By draft head is meant the vertical distance between the center of shaft and the tail water for horizontal wheels and between the center of the guide buckets for vertical turbines, as B in Fig. 327.

This table gives draft heads slightly too small for turbines under steady loads and working at full gate and too great a head for rapidly fluctuating loads and partial gate.

Draft tubes should always be conical. The proper diameter for any particular draft tube may be figured, using the above table as follows: A conical draft tube must not be more than 12 feet in diameter, at a height of $\left(10.5 - \frac{V^2}{64.4}\right)$ feet above tail water, where $V = .285\sqrt{64.4\,H}$.

Short and small tubes must dip into the tail water 6 inches to 12 inches and 20 inches to 24 inches for long and large tubes. A greater dip permits a greater draft head within limits.

Where heads fluctuate badly and a sensitive governor is used, the wheels should set close to tail water to avoid oscillations.

Even where the turbine sets below tail water a draft tube increases the efficiency.

Frequently with large draft tubes when running at part gate the draft tube does not fill with water at all, and the turbine runs under a severe loss of head. Draft tubes are generally made much too thin, 3/16 inch is a common thickness, while good practice would be nothing thinner than ¼-inch. The velocity of the water amounting to about twice the peripheral speed may be calculated from $V = .285\sqrt{64.4\,H}$, where V is velocity in feet per second and H is the total head acting on turbines in feet.

All turbine settings should be provided with a gauge, A, Fig. 345, to indicate the pressure at turbine due to the head, and a vacuum gauge B to indicate the vacuum in the draft tube due to the draft head.

Throttling gates should never be used to regulate the speed, on account of the great waste of power.

All turbine makers supply books giving dimensions of flumes for different sized wheels and tables of speed, power and quantity of water for different sized turbines under various heads.

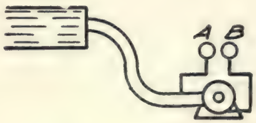


FIG. 345.

Whether the turbine chamber is of wood, masonry or steel, the water should be admitted to the wheel and conducted away from it at a velocity of about 80 feet to 90 feet per minute.

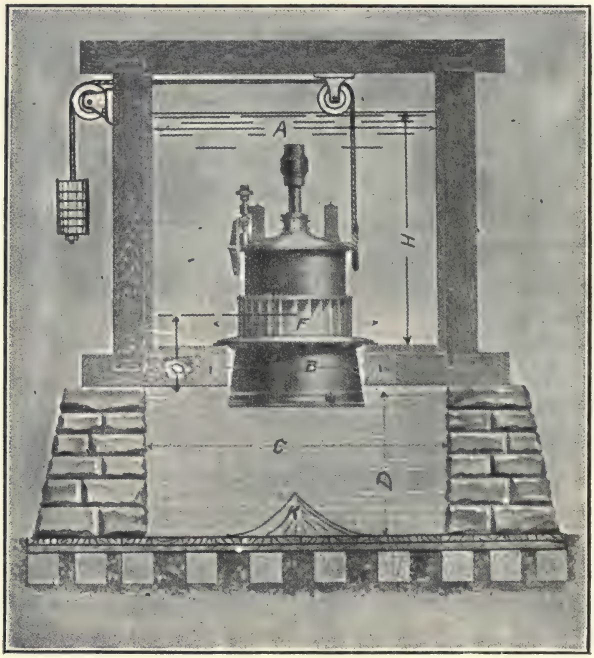


FIG. 346.—Setting for cylinder gate turbine.

Of course, if to gain this condition necessitates an expenditure of more than the head gained is worth, a higher velocity is advisable. By referring to the tables giving the water used by

the selected wheels and dividing this quantity of water by 80 or 90, the proper area of the wheel chamber, that is $A\ H$ (Fig. 346) is obtained. The area $C\ D$ should also equal $A\ H$ where practicable, though often the added cost of a deep wheel pit will not warrant a lower velocity than 100 feet per minute.

The effective depth of tail water should be taken as $D-E$. Where the desired depth is not attainable, a diffuser K, or a concrete diffuser should be used.

The tail race is not usually continued down stream at the depth D, but is gradually shallowed and widened so that the

FIG. 347.

area $A'\ B' = A\ B$, Fig. 347, the race discharging into the river on an angle.

As the speed of a turbine grows less with the increase in size, two or more small turbines may be mounted on the same horizontal shaft. There is a further advantage in having several small wheels rather than one large one, in that there is less trouble with the draft tubes.

The floor upon which the turbine rests must be perfectly unyielding. It supports not only the water but also the turbine. Whenever the depth of water over turbines will permit, set the wheels high enough above tail water so that a man can pass under the floor of the flume, and by removing a man-hole

cover provided for the purpose in the draft tube, adjust the step of the turbine (Fig. 348). Some form of step adjustment

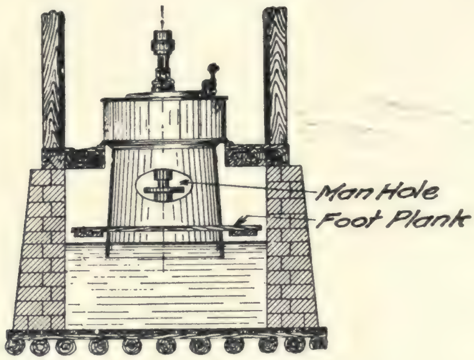


FIG. 348.

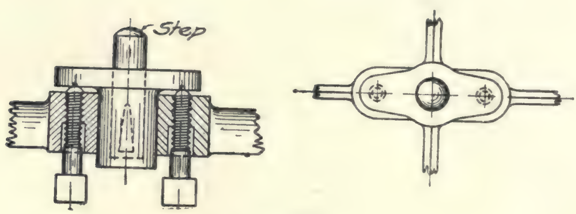


FIG. 349.

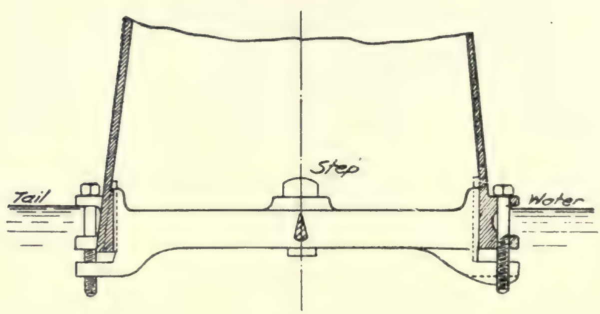


FIG. 350.

other than that usually used should be designed. Figs. 349 and 350 are given as suggestions.

THE PELTON WHEEL.

Pelton is the name commonly given to that form of water wheel which receives its water power from the force of one or more jets of water directed against the numerous cup-like vanes situated around its periphery. This wheel is also called the hurdy-gurdy wheel, or tangential wheel.

Fig. 351 shows a Pelton mounted in an iron frame. The water after leaving the vanes drops down to tail water utilizing none of the fall from the bucket down. Thus on a low head, especially when there is liability of back water, a serious proportion of the head is lost.

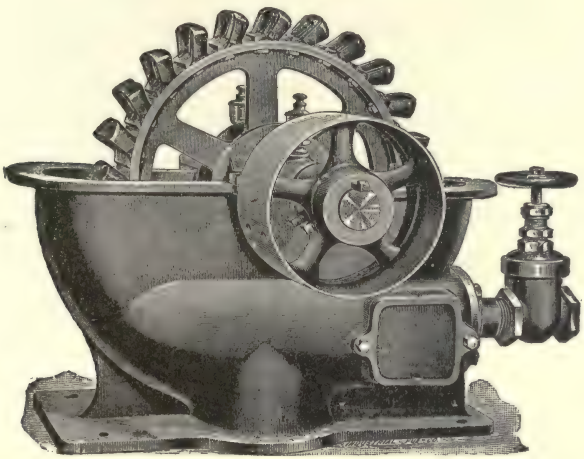


FIG. 351.—Pelton water wheel.

As now constructed, the Pelton uses a nozzle so small that the power derived from it is quite insignificant comparatively. To increase the power of the Pelton, the velocity being fixed by the machinery they are to drive, two expedients are resorted to.

One. The diameter of the wheel may be increased to as great a diameter as required to give the power. 33 feet is the greatest diameter in use; 12 feet is rather uncommon, and six feet is the largest *standard* size.

Two. The number of nozzles may be increased. The Pelton Water Wheel Company built a quintex Pelton or a five-nozzle

wheel. Again for any large units more than one wheel may be mounted on the same shaft. Of course, all this complicates the plant and there comes a time when the complication makes the high pressure turbine preferable to the Pelton. For heads up to about 150 feet and for powers of over 500 h.p. the turbine has as great an efficiency, is cheaper, and is preferable to the Pelton. For higher heads up to 300 feet and large powers the field is open to both the Pelton and the turbine, but above this head the Pelton begins to rapidly out-distance the turbine in point of cost and efficiency. There are quite a number of plants in operation where a head of from 1200 to 1600 feet is utilized successfully.

Tables giving the sizes and powers of the standard Pelton may be obtained from the manufacturer. In these tables the " effective " head is given, that is, the vertical distance between surface of the head water and the point where the water strikes the vanes, and not the distance down to tail water as for the turbine.

As will be seen from Fig. 351 the weights of the revolving parts are quite light and therefore add very little to the regulation of speed under a fluctuating load. It is therefore quite necessary to use a heavy fly wheel where good government is essential. In many large plants the weight of the generator-armatures or fields is depended on. Where the generator is of the revolving field type this may be sufficient.

For high efficiency the velocity of the water issuing from the nozzle must remain at a maximum. The Pelton Water Wheel Company regulate their wheels in four different ways.

(1) By deflecting the nozzle so that the water does not hit the vanes.

(2) A cut-off hood is placed in front of the nozzles by means of which the discharge area of the nozzle is varied.

(3) A deflecting plate is used to deflect the water.

(4) A plug nozzle is used. In this nozzle a tapering pin or needle is placed like the valve of a steam, water injector, the operation of which regulates the quantity of discharge without materially affecting the velocity. The Doble needle regulating nozzle (Fig. 366) is the best on the market. It is made by Abner Doble Co., San Francisco, Cal.

Any form of water wheel governor may be used with the Pelton, either to deflect the nozzle or to alter the area of discharge.

An excellent plan where for certain periods the power is greatly reduced is to have several nozzles, any one of which may be shut off by hand, leaving, say, one nozzle to carry the reduced load This gives the maximum efficiency.

REGULATION.

Long pipes carrying water for power purposes are subject to great abnormal pressures due to the quick shutting off of the water from the turbines.

The water under motion has acquired a certain amount of momentum which is proportional to the product of the weight and velocity. To arrest this momentum requires power or some means must be provided for the escape of the energy. In the early history of the development of water powers under high heads, some of our best engineers met with serious accidents such as the bursting of huge steel pipes and the flooding of power houses; but now that the agents of destruction are known, provision is made for their subjection.

To secure the best regulation, what is known as an open setting should be approached as nearly as the conditions will permit. That is, the conditions secured in the design for the power house at Nobelsville, Ind. (see Figs. 352 and 353), where the water stands directly over the turbines with no appreciable loss of head at any point. The Yorktown plant (Fig. 300) is also an example. Where a long penstock is used, this condition may be approximated by providing a small reservoir at the outlet of the penstock. The size of this reservoir will depend on the sensitiveness of the governor, and should equal the amount of water the penstock will carry in twice the time necessary to close the turbine gates. If the gate is left closed the reservoir will run over unless its level is above the inlet of the penstock and its capacity about equal to that of the entire penstock. Usually a standpipe made of steel, see Fig. 354, of only a few feet diameter is used, in which case the water immediately runs out at the top, when the gates are quickly shut. This relieves the pressure on the pipe line and the water stored in the pipe under the increased head supplies the succeeding demand for power while the water is getting up to speed again.

The overflow of the standpipe should be on a level with the surface of *high* water at the inlet of penstock, unless the fall

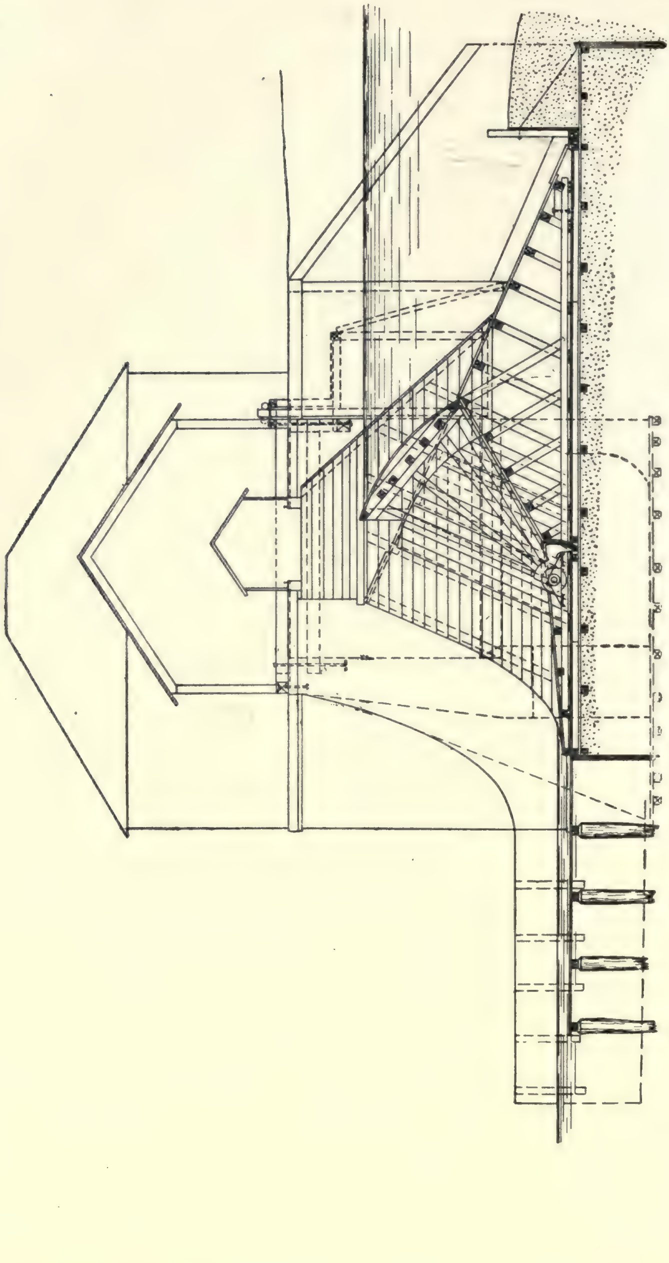


FIG. 352.—Dam and Tainton gate.

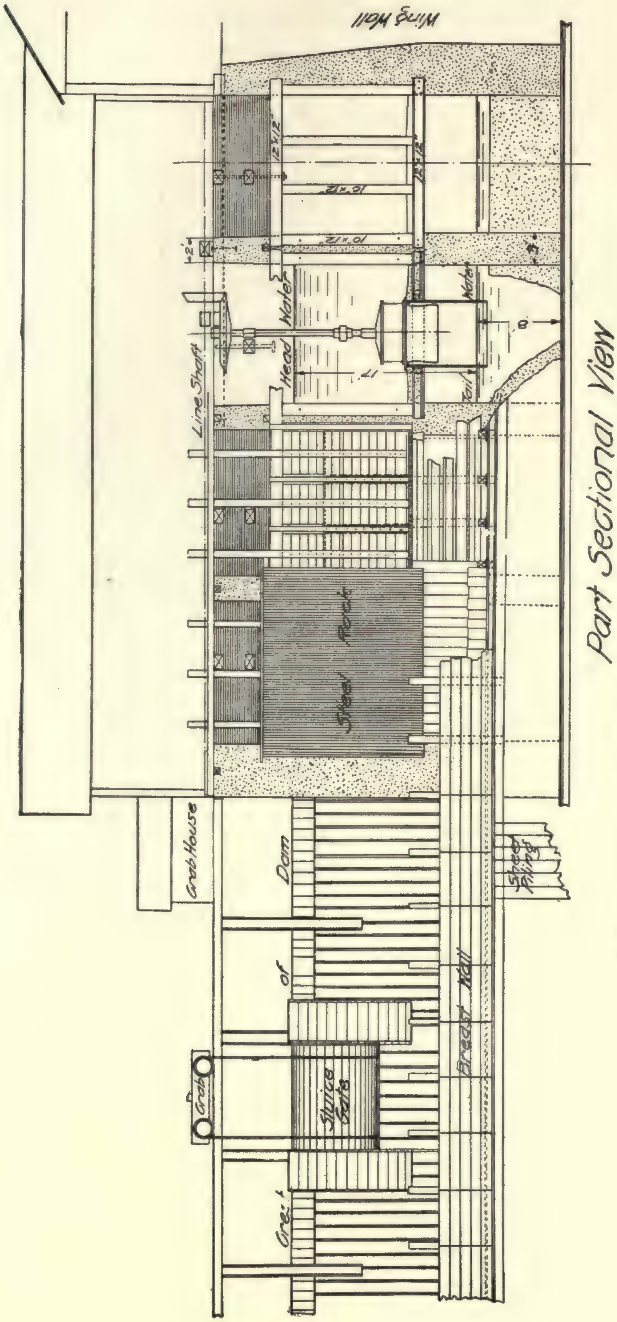


FIG. 353.—Looking down stream at power-house and dam.

in the line is so great that the pressure on the penstock (head H, Fig. 354), would be excessive when the standpipe is filled. In this case the standpipe would give place to safety valves along the line of penstock. Standpipes must be protected from freezing. In Fig. 355, c is a standpipe with a tank at the upper end and large enough to take care of the fluctuating water.

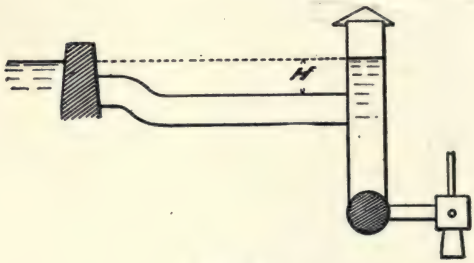


FIG. 354.

Safety valves similar to those on a steam boiler are built. They should be able to discharge, when open, the full flow of the penstock.

Fig. 355 indicates the location of the safety valve at a. This should be so placed that the overflowing water will escape into the tail race. Where there is a high place in the penstock air valves must be placed as at b, to prevent the formation of air plugs, or, where the height is not too great, a better plan is to place standpipes at these points. Galvanized sheet iron will often serve the purpose as at b'.

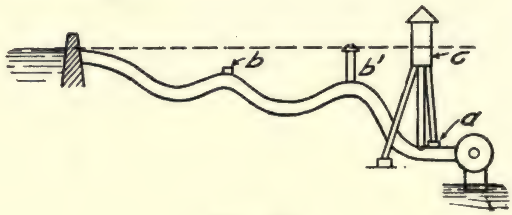


FIG. 355.

The ordinary steel standpipe such as is used by city water works and here shown at c, Fig. 355, makes an excellent standpipe. It must be heated in winter to prevent freezing.

Fig. 356 shows a good design for a concrete-steel stand pipe.

Figs. 357 and 358 show two types of cheap governor made by the Woodward Governor Company. These are considered by the author to be the best cheap governors on the market.

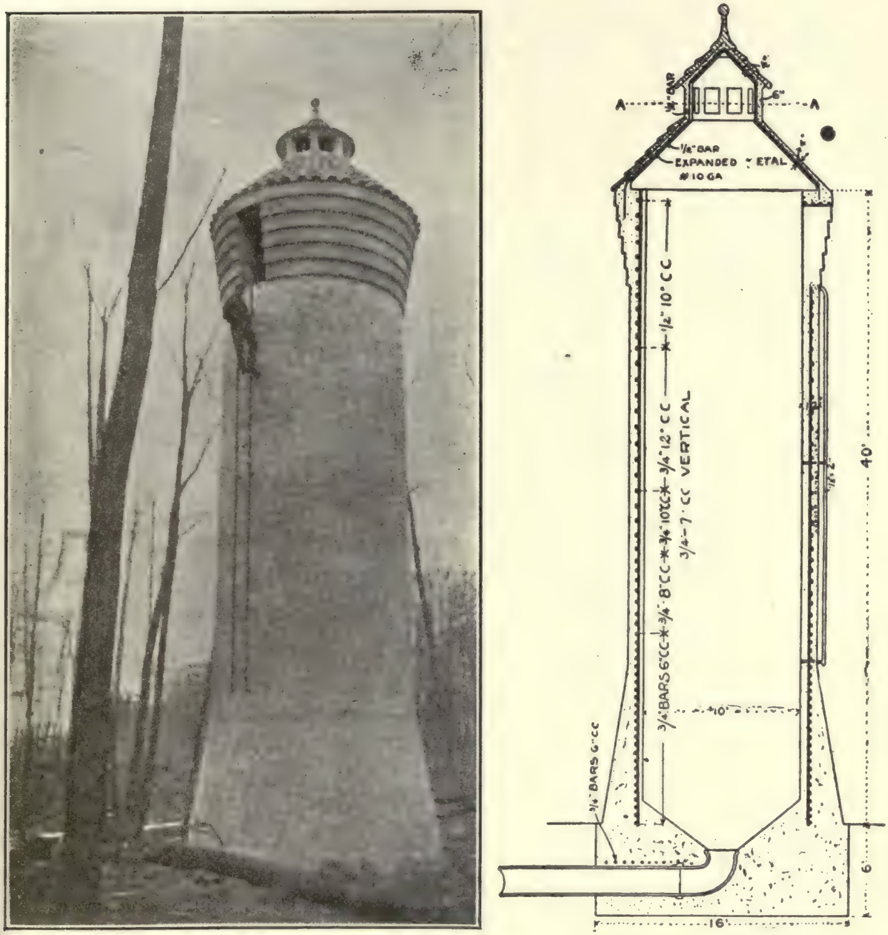


Fig. 356.

Governors.

The selection of governors should first be made after a thorough study of the situation. Sensitive governing means severe wear and tear on the gates, and should be avoided where pos-

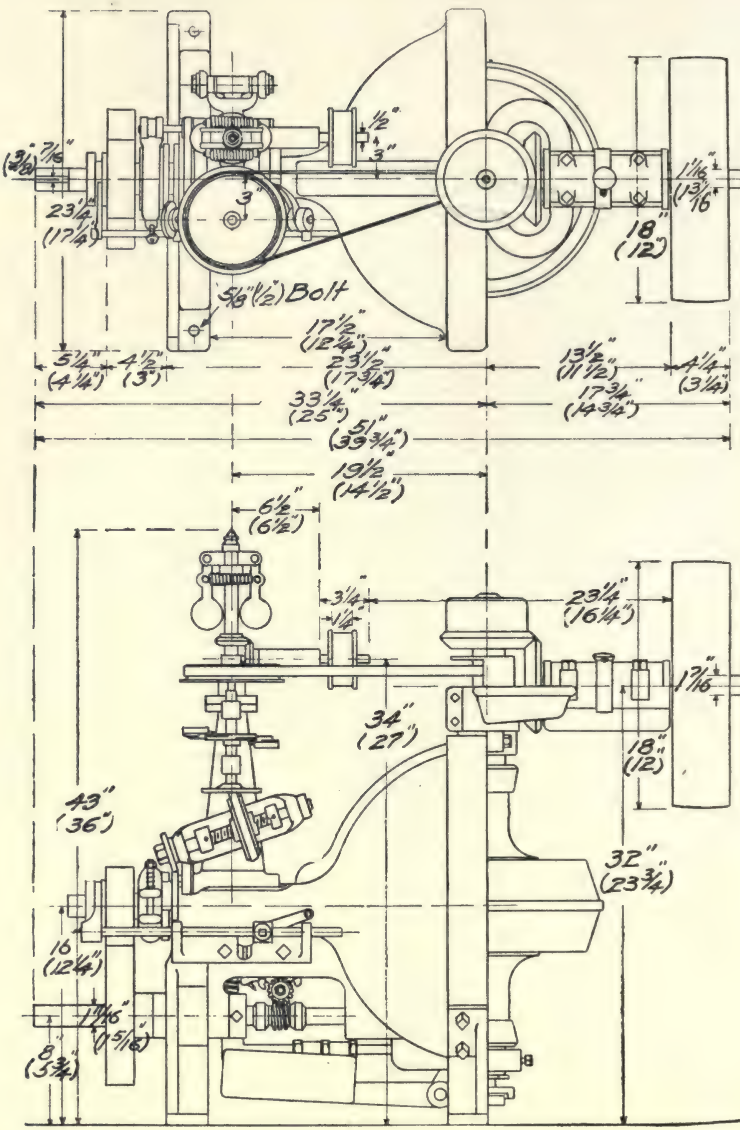


FIG. 357.—Woodward governor, Types C and D. (Dimensions in parentheses are for Type D.)

sible. Many lighting plants are successfully operated where it requires 20 seconds to close the gates and while 20 seconds may be the extreme, it has been the author's experience that for all but the most important, or special plants, 5 seconds is quite satisfactory.

The approximate energy necessary to operate turbine gates, which are properly balanced and installed, is given in Table XL. Where it is desired to obtain the energy necessary to operate gates which are already installed, the test can be made

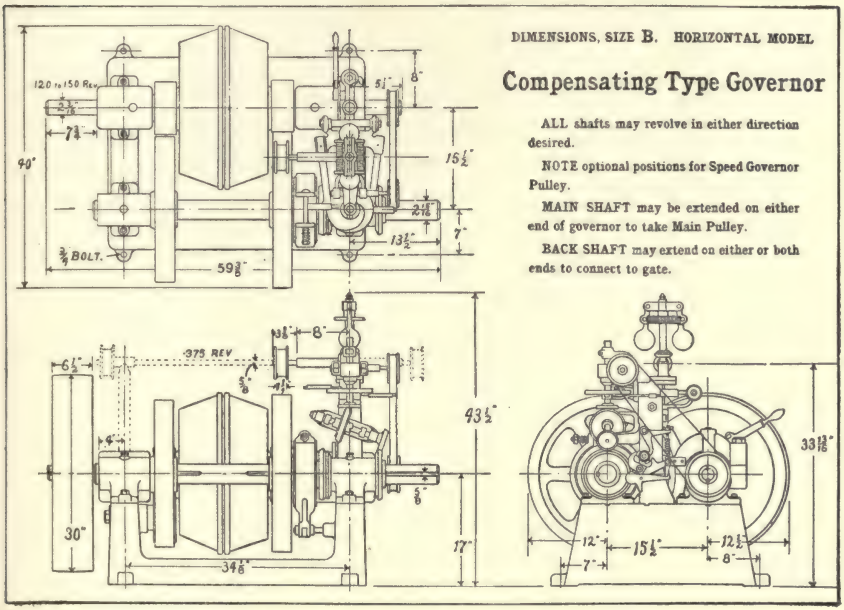


FIG. 358.—Horizontal Woodward governor.

by turning a hand wheel of known radius through the medium of a spring balance. The test should include a measurement of the pounds necessary to start the gate; to move it when 0.25 open; to move it when 0.5 open, and to move it when 0.75 open. From these measurements the average foot pounds is found by multiplying average pull in pounds by the product of the circumference, in feet, of the hand-wheel and the number of turns necessary to close the gate.

A powerful governor is one which will in a given number of

seconds from rest bring up to a high velocity a mass of iron sometimes weighing several thousand pounds. Every moving part connected to the gates must be set in motion regardless of friction.

In the case of cylinder gate turbines the cylinder *c* is counter-

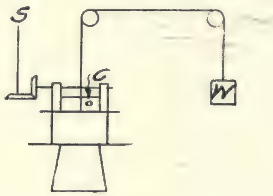


FIG. 359.

balanced by means of a weight *W* (Fig. 359). The governor must therefore move the weight and cylinder as well as the gears, shafts, etc.

The turbine having wicket gates as in Fig. 324 is balanced by water pressure so does not require a counter-balance. There

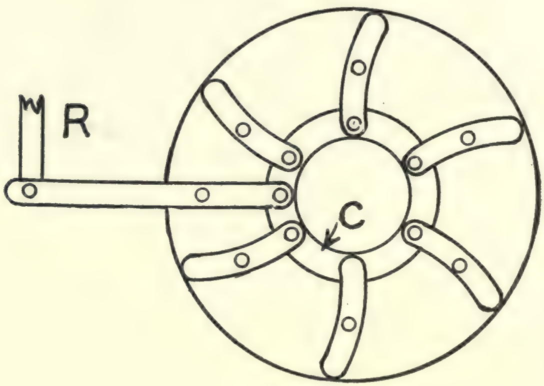


FIG. 360.

are more parts to get out of order than with a cylinder gate, but the regulation may be more perfect and requires a less powerful governor.

In Fig. 359 the governor operates the gate stem *S* only, which may be reciprocated or revolved to suit the conditions.

Fig. 360 shows a plan view of a wicket gate turbine. Until

recently the collar C was operated by gear wheels and a rod running to each gate, but in the present form of Leffel and Smith turbines the parts are reduced in number and also the weight. The governor works the draw bar R to cause the rotation of the collar C.

All bearings for the various parts of turbine gates should be of bronze and of liberal proportions.

TABLE XL.

FOOT POUNDS REQUIRED TO OPERATE TURBINE GATES.

H.P. of Turbine	Head of Water.										
	10	15	20	25	30	35	40	50	60	80	100
25	400	360	330	310	290	265	250	220			
50	700	630	580	540	500	470	440	390	350	310	
75	1000	900	830	770	720	660	625	550	500	440	400
100	1200	1080	1000	935	860	800	750	665	600	530	465
150	1600	1450	1320	1240	1150	1060	1000	880	800	700	640
200	2000	1800	1660	1550	1440	1330	1250	1110	1000	880	800
250	2400	2170	2000	1870	1730	1600	1500	1330	1200	1060	930
300	2800	2530	2330	2180	2000	1870	1750	1550	1400	1240	1100
400	3600	3250	3000	2800	2600	2400	2250	2000	1800	1600	1400
500		3900	3600	3360	3120	2880	2700	2400	2160	1920	1680
600		4500	4200	3920	3640	3360	3150	2800	2520	2240	1960
800		5800	5400	5000	4650	4300	4000	3600	3240	2880	2500
1000				6000	5580	5160	4800	4500	3900	3450	3000

Location of the plant should influence the selection of a governor and where the plant is in some isolated place, the less complicated the governor, the better. Of course extra parts could be kept always on hand where the item of expense is second in importance to good design. Again if the superintendence of the plant is to be given to incompetent men (a common and costly experiment) the less complicated the governor the better. Throttling valves should never be used to regulated turbines.

Fig. 361 shows a standpipe with a damping pipe a. The object of this pipe is to prevent the water oscillating in the standpipe under the action of the governor. The one shown

is three feet in diameter and connects to the standpipe one foot above the surface of the water in the distant reservoir. For heads above 100 feet or so the standpipe loses part of its efficiency on account of the inertia of the mass of water it contains. The cost also becomes prohibitive and recourse is had to safety valves and heavy fly-wheels. In fact, all turbines, where good regulation is desired, should be provided with them.

These fly-wheels range in weight from 5000 pounds to almost any weight, and peripheral speeds of from 5000 to 10,000 feet per minute, and are made of built-up plates, or cast steel with steel tires shrunk on. Fly-wheels serve to take care of all the

FIG. 361.—Stand pipe with dampening pipe.

smaller and more rapid fluctuations due to surging in the penstock, draft tubes or changes of load. The governor does not have to act so quickly and the pressures on the penstock are more easily controlled and are not so severe. Vibrations of the foundation are minimized and most of the noise eliminated.

For fluctuating loads the draft tubes should be short, as severe oscillations are set up in long tubes tending to cause vibrations, uneven power and even to damage the turbine.

The best governors on the market are the Lombard, the Improved Sturgess, the Woodward and the Replogle. The first three makes, which use energy stored in tanks to operate the gates, are called hydraulic governors, while the Woodward and Re-

plogle which operate the gates by friction derived from inertia balls similar to those on a steam engine governor, are called mechanical governors.

All governors are set into operation by the change of speed in the line shaft. A pulley on the line shaft drives the friction cones, and a second pulley drives the governor balls. A change of speed then varies the position of the balls which causes the friction cones on the gate shaft to operate, or causes the hydraulic piston to act on the friction cones. It will thus be seen that the speed has to change perceptibly in order to actuate the governor, a fact which of itself makes perfect government impossible.

It would seem to the writer that the first word " go " should come from the switchboard rather than from the line shaft. It is the fluctuation in the power that brings about the change of speed which we wish to avoid. Therefore if a special ammeter sets the governor in motion a full second will be clipped off of the time taken by the best governors in starting the gates. The inherent difficulty in attaining perfection in water power government is the impossibility of setting in motion the dead water in the pipe or flume the instant the gate is opened.

However, if the ammeter is used to start the governor, all well-designed water powers may be made to regulate with a variation in speed of less than three per cent. on throwing off 75 per cent. of the full load, or two per cent. under changes amounting to 50 per cent. full load. There is a limit to the speed with which a gate can be closed, due to mechanical reasons, 1 to 1½ seconds is about this limit. Therefore when a second is lost in getting the initial impulse to the governor it is a serious loss.

The curves, Fig. 362, show the action of a governor when the load is suddenly increased from 25 kw. to 75 kw. These curves are taken from Mr. M. A. Replogle's paper read before the American Society of Mechanical Engineers, May, 1906.

Mr. John Sturgess gives a set of curves which are exactly like Mr. Replogle's in showing that valuable time is lost in starting the governor. In these curves it will be seen that the speed does not increase perceptibly for fully 0.25 second. This is due to the inertia of the moving machinery and shows the importance of providing a fly-wheel effect where possible. It

also shows that this effect retards the action of the governor at the start, though it helps it in the end. Hence with a governor started from the ammeter we will have the governor acting 0.25 second to 0.5 second before the speed varies at all and in another second the gates will be wide open and, if a proper fly-wheel is provided, the regulation will be practically perfect.

All governor experts are agreed that the cylinder gate turbine, especially those with the lip, *b*, Fig. 363, are the most difficult

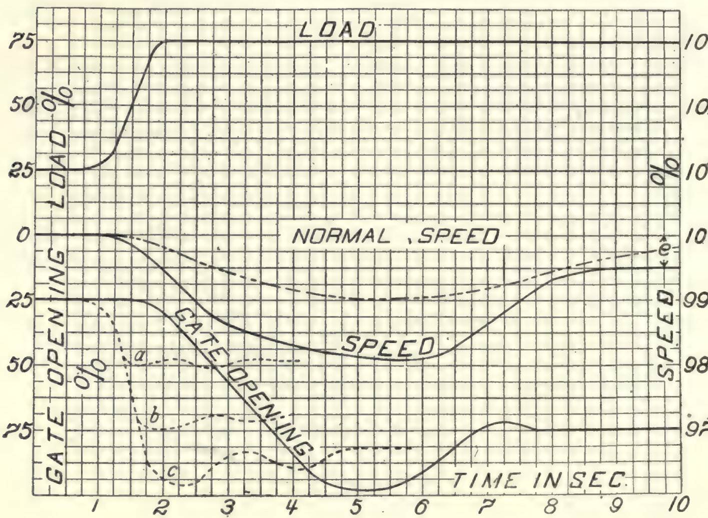


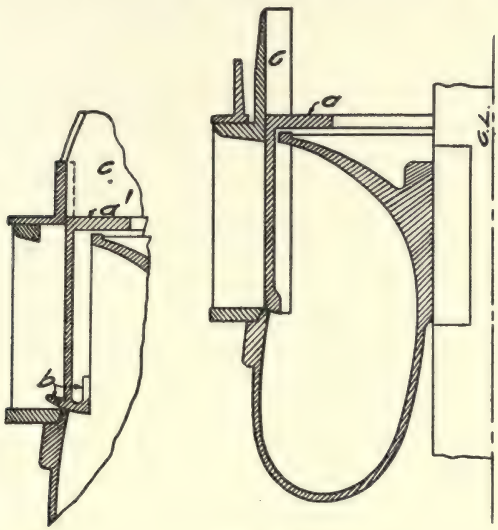
FIG. 362.—Curves showing action of governor.

to govern. There is a general tendency now to build wicket gate turbines. In Fig. 364, *c* is a slide for the cylinder gate when the turbine is of the horizontal type. Without this the gate is apt to bind. Another defect in the cylinder gate is its poor part load efficiency and the only advantage the writer knows of is that it is more nearly water-tight when not running than is the wicket gate turbine.

In the case of wicket gates it is common practice to so balance the gates that there is, at all times, a tendency to close. This is so that in case of accident the gates will stop the wheel. This is undoubtedly a mistake. It makes governing more difficult,

subjects the governor to unnecessary load, and serves no useful purpose. All well-designed plants have some means of shutting off the water and in case anything did break, the penstocks, or flumes, could be emptied. Water wheels cannot quite double their normal speed when racing, and therefore the danger from the centrifugal force bursting the fly-wheel or armature is not great. With perfectly balanced wicket gates, short vertical draft tubes, proper fly-wheel effect, and an open setting, we need have no difficulty in obtaining, practically, perfect government.

The Lombard governor Company now build a type "Q"



Figs. 363-364.

governor which possesses a new feature of great merit. The gates are closed in the usual way, but the valves are so arranged that they do not completely close at first, an opening of about one inch being left to prevent water hammer.

Summing up, it appears that a governor should possess the following features:

(1) The first impulse tending to set the governor into action should come from the ammeter on the switchboard.

(2) The governor should be so arranged that the speed may be increased, for synchronizing, from the switchboard.

(3) Where the energy required to operate the gates is great, a governor of the hydraulic type should give the best service.

(4) The gate moving should be of a differential character so that the mechanism will not be subjected to excessive strains.

(5) If the governor is set in motion by the ammeter it should instantly operate on a variation of the load, but if of the types now on the market, the governor should commence acting on a variation of the speed of the line shaft of ½ per cent. from normal.

(6) The governor should move the gates through their full range within a certain time, say 1.25 to 15 seconds.

(7) Under steady head and load the governor should remain stationary, and under all changes of load it should not make more than three movements in readjusting the gate to the new load condition.

(8) The governor should develop a definite amount of energy.

(9) All the important bearings should be of the self-aligning self-oiling type.

In the above, clauses 1, 3 and 5 are by the author; 2, 4 and 9 are by Replogle; 6, 7 and 8 are by Sturgess.

Referring to Fig. 362, the dotted line shows the gate action when the impulse to act comes direct from the ammeter. The gate, commencing to open gradually, gets up speed and then approaches full open with a retarded motion at which point the governor begins to be acted upon by the change in speed of the line shaft (if there is a change in speed), and assumes control. The special ammeter is so adjusted that if the change of load is not severe the gates will increase the opening ⅓ as at *a*, and then wait for the governor. If quite severe they open ⅔ as at *b*, and for unusually heavy changes the gates are moved to 9/10 opening. In case of loads being reduced the above operation would, of course, be reversed.

In Fig. 362, *e* is the permanent "set," of the regulation and indicates lack of proper design. If the turbines are properly proportional in speed and power this drop will not take place unless the penstock is long and small, and not provided with a standpipe in which case it may take a minute or more for the water throughout the entire pipe line to get under full motion.

If we could pre-determine the curves shown in Fig. 362, we could easily design the fly-wheel to store up the energy necessary to prevent the fall in speed shown.

The work E given out by the fly-wheel, while the velocity falls from V_1 to V_2

$$E = \frac{w\,(V_1^2 - V_2^2)}{2\,g}$$

where w = the equivalent weight of all revolving parts in pounds; V_1 = velocity in feet per second of the weight moving around in a circle of an average radius and at the maximum speed; V_2 = the reduced speed in feet per second of the revolving weight.

Therefore if we take as an example the curves in Fig. 362, we find that the average drop in speed for eight seconds is 1.2 per cent.; so, if the normal speed is 10 revolutions per second, the average drop is .12 revolutions per second for eight seconds. To carry the added load of 50 h.p. for eight seconds the fly-wheel must give out $50 \times 33{,}000 \times 8/60$ foot pounds of work per second = $220{,}000 = \frac{w\,(V_1^2 - V_2^2)}{64.4}$. After getting the mean radius of the armature, shafts and pulleys, calculate the fly-wheel effect. Say that this is found to be 20,000 foot pounds per second, then $220{,}000 - 20{,}000 = 200{,}000$ foot pounds to be supplied by the fly-wheel. If the mean diameter of the fly-wheel rim is 12 feet and the maximum revolutions per second of the fly-wheel is 10, then

$V_1 = 12 \times \pi \times 10 = 377$ feet per second; and

$V_2 = 12 \times \pi\,(10 - .12) = 372+$;

then

$$200{,}000 = \frac{w\,(377^2 - 372^2)}{644} = 3420 \text{ pounds.}$$

Therefore a fly-wheel having a rim weighing 3420 pounds would have prevented the fall in speed. Of course this would be a very small fly-wheel, but in this example the fall in speed was slight and the load small.

Fig. 365 shows an ideal arrangement of standpipe and turbine unit. The penstock is continued on through the power house and ends in the reservoir.

Mr. G. A. Buivinger describes such a plant. Much difficulty had been experienced with this power plant due to long penstocks (6200 feet by 8 feet), long draft tubes, etc. A 10,000

pound fly-wheel was placed on the shaft, and while this aided the regulation, it did not perceptibly effect the pressures on the pipe. Finally a reservoir 50 feet in diameter and 12 feet deep was built as shown. The capacity of the power plant was 800 horse power under 47-foot head, and this reservoir supplied 2000 cubic feet of water for each foot in depth and therefore one foot of water supplied $2000 \times 62.5 \times 47 = 5{,}875{,}000$ foot pounds per minute and for 30 seconds twice that, or 11,750,-000, which is about half the full load output of the plant.

This arrangement gives the water in the penstock a free path into the reservoir. The reservoir can frequently be built of

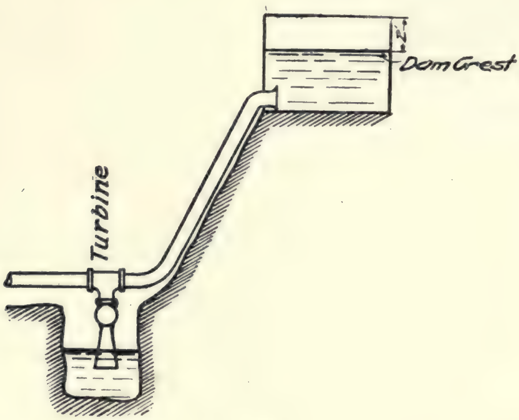


Fig. 365.

concrete on top of a near-by cliff, and in such cases (where no tower is required) there should be no limit to the height. The efforts made to reduce the pressures in the above plant by installing a 10,000 pound fly-wheel would indicate that the relation between pipe pressures, fly-wheel and regulation is not well understood. Water hammer can not be prevented by the use of a fly-wheel and the present day governor. The governor acts from the line shaft and the fly-wheel retards the action of the governor. The presence of the fly-wheel would, other things being equal, permit the more gradual opening of the gates, but owing to the fact that it takes longer to get the line shaft up to speed again with a fly-wheel, if the gates operate slowly the result will be that the speed will not fall so

low but the period will be increased and where the load fluctuates rapidly the second peak may come before speed is restored. Therefore it is fully as important to have quick gate action with a fly-wheel as it is without. The dash and dotted line above the speed curves illustrates the effect of a fly-wheel.

The only safety from water hammer is the standpipe or relief valves.

The function of a standpipe is not to take the place of the fly-wheel, its duty being to prevent water hammer.

Water flowing in a long penstock must be suddenly arrested, the doing of which produces a bursting tendency. If we let P equal the normal working pressure per square inch on the penstock, due to the hydrostatic head; P_1, the pressure which will be produced by shutting the gates a certain percentage of the opening, v = the velocity in feet per second in the pipe at the time the gate is moved. Then

$$P_1 = \tfrac{1}{2}(P+62.92\times v)+\sqrt{\tfrac{1}{4}(P+62.92\times v)^2-62.92\times P\times v\times p}$$

p = the percentage of the normal output remaining after a reduction of load. Thus if the wheels are running under full load and half the load is thrown off, $p = .50$. In the above, the penstock is not supposed to increase in volume under the pressures.

Governing High Head Systems.

In the development of power from water under high pressure, certain difficulties arise which must be carefully considered, else disaster will follow. As already explained water hammer is a great danger to the pipe line. It has been stated in the preceding pages that safety valves and standpipes were the two methods employed to prevent water hammer. The following refers to heads above 200 to 300 feet where standpipes cannot be employed.

Where tangential wheels are used and the head too great for a standpipe, the deflecting needle nozzle serves to prevent water hammer and gives the highest efficiency where the state laws require that the natural flow of the stream be uninterrupted. Here the waste of water can not be avoided, but where it is possible to store the flow during light loads great economy is desirable. In this case the needle nozzle shown in Fig. 366 gives the highest efficiency and the best regulation. The force required to operate this nozzle is very slight.

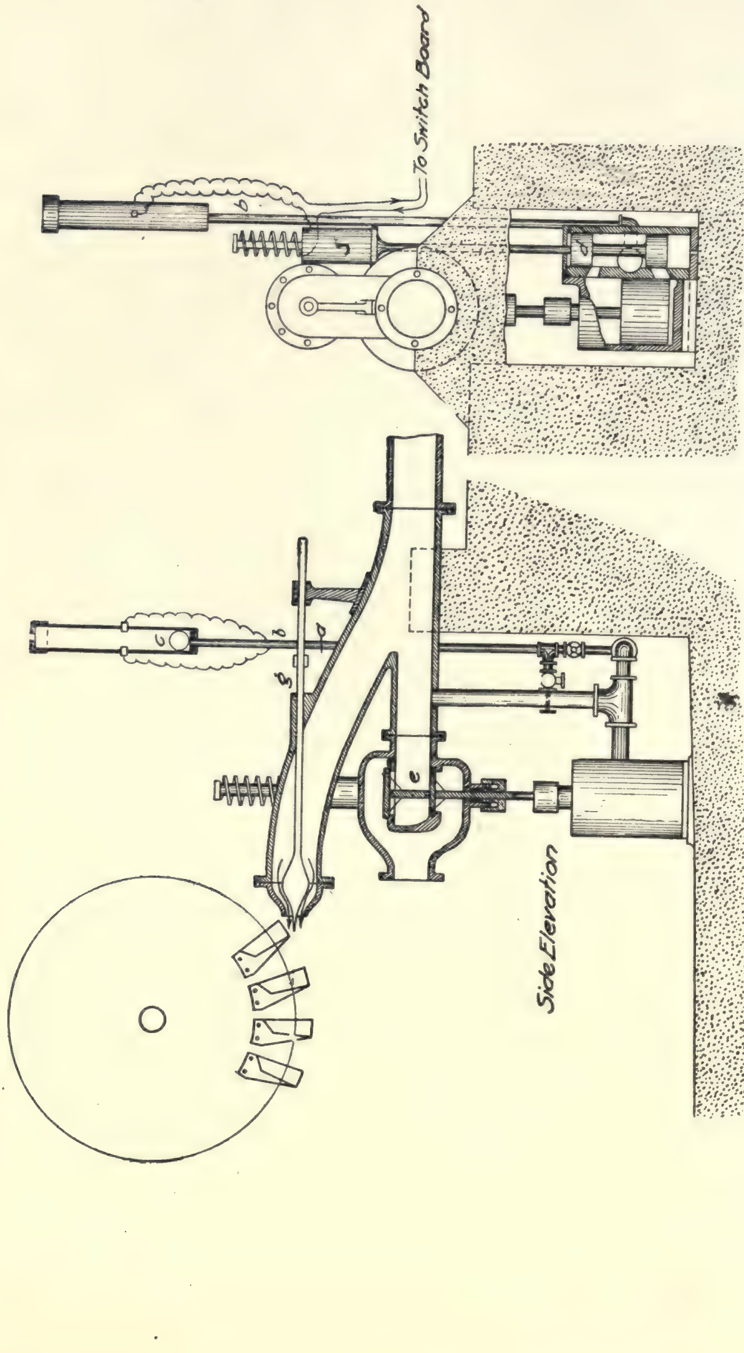


FIG. 366.—Safety valve for pipe line.

It is evident that when the needle is suddenly thrust outward water hammer is produced. To relieve this excessive pressure safety valves must be provided and as there is no more important part of the power equipment it should be carefully designed. The common type is built similar to the safety valve on a steam engine and is open to the danger of sticking in the seal. This failure to operate at the critical time has caused the loss of many thousands of dollars. To be safe against sticking, the valve should be made to rotate constantly, or constructed as in Fig. 366.

In this safety valve water stands normally at some level, *a*, in the pipe, *b*, compressing the air in *b*. As the pressure increases the water rises in *b*, till the hollow ball *c* floats up and completes the circuit between the switchboard and the solenoid *S*, thus causing the balanced valve *d* to descend and the valve *e* to open.

Instead of operating the valve, *e*, this same arrangement could be used to deflect the nozzle. The governor operates the needle at *g*.

TESTING.

There can be no object in insisting on a guarantee for a machine unless a test is performed to ascertain whether the guarantee has been made good or not. Such tests cost a good deal and it is seldom indeed that large turbines are tested after being guaranteed and placed.

However, if a test shows up a loss of a few per cent. on, say, a 500 h.p. turbine, and the water wheel company can be held for this loss, the amount saved will more than pay for the investment. For a deficiency of five per cent., 25 h.p. would be lost, which at $20 means $500 clear cash lost each year and at ten cents per kw.-hr. (3000 hours per annum), would mean $5595.

The turbine manufacturers send their wheels, that is, one of each pattern, to Lowell or Holyoke, Mass., where there are companies that make a business of such work. However, the engineer in charge of the power plant should from the start plan to make a test, because the testing flume efficiency is seldom attained in practice.

During construction, while the wheel pit is free from water, a weir *W*, Fig. 367, should be put in. It is often placed

under the power house, but usually it may be placed further down stream, the greater the distance from the turbine the better. This weir should be so constructed that after the test it can be easily removed. If it is necessary to put the weir close to the wheels, an equalizing rack is placed parallel to and at distance of five or six feet from the weir. This rack has numerous small openings equal to about one-fourth or one-fifth the entire area.

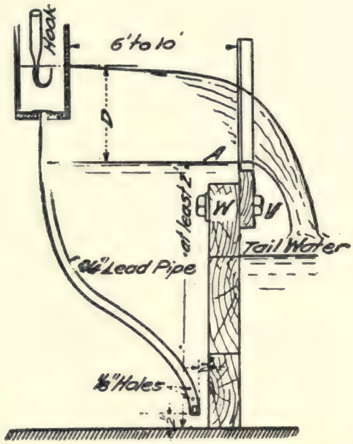


Fig. 367.

Fig. 367 shows how the exact depth over the weir is obtained by means of a hook gauge and a water-tight box communicating through a ¾-inch lead pipe with head water. As shown this gauge may be placed above the equalizer and it will give the same reading as if placed below. Air must be admitted under the overpour at V, otherwise the formulas given in Chapter II will give too large a flow.

Fig. 369 shows the general arrangement of the friction brake as arranged for testing a vertical turbine. If the friction-pulley is heavier than the gear ordinarily used, it is suspended by means of cords passing over pulleys and attached to counter weights. This is a refinement which would not be necessary

in ordinary tests. The brake is suspended in the proper position around the pulley by means of ropes and weights W'.

A bell crank C is used to transmit the turning effort from the brake arm to the scales (Fig. 369). The operation is best demonstrated by an actual example. Let the dimensions of the bell crank C be $M = 5$ feet and $N = 6$ feet; the effective length of the brake lever 0, Fig. 370, be 10 feet. Then the total leverage acting on the friction-pulley

$$= O \times \frac{N}{M} \text{ or } 10 \times \frac{6}{5} = 12 \text{ ft.}$$

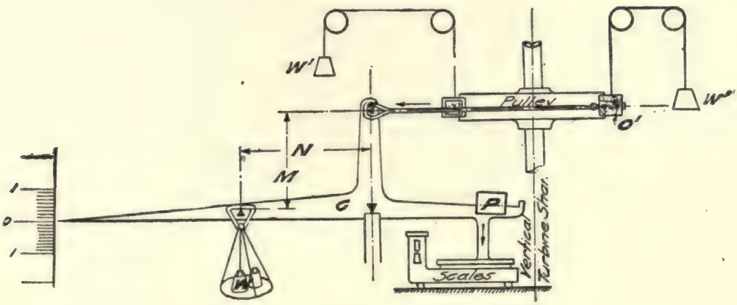


FIG. 369.

and the total force acting on the pulley face is

$$W\,O\,\frac{N}{M} = 10\,\frac{6}{5}\,W = 12\,W.$$

wherein W is the weight as measured at the end of the bell crank arm N.

The weight, W' is used to counter-balance the brake rigging.

$$\text{The power of the turbine} = \frac{6.28 \times O \times W \times \text{r.p.m.}}{33{,}000} = \frac{6.28 \times 12 \times 200}{33{,}000}$$

$= 457$ h.p. where $6.28 = 2\pi$, $O =$ effective inches, $W =$ weight on scale arm.

At the start everything must be in balance. As shown in Fig. 369 the bell crank is the heaviest to the left of the knife edge, and in practice an arm P, would be attached to balance

the scale pan, etc. The friction pulley should have about 100 square inches frictional surface per h.p., and may be from 18 inches to 120 inches in diameter. Heavy cylinder oil may be used as a lubricant, however, green pig fat is the best.

During the test the exact head is obtained by getting the level of the water over the wheels. Then, having the quantity of water per minute the i.h.p. is found from $\frac{\text{Head} \times 62\frac{1}{3} \times Q}{33{,}000}$. The h.p. measured with the brake, divided by the i.h.p., gives the efficiency.

Undoubtedly one of the best all-around dynamometers is that shown in Fig. 370. This dynamometer has been thoroughly

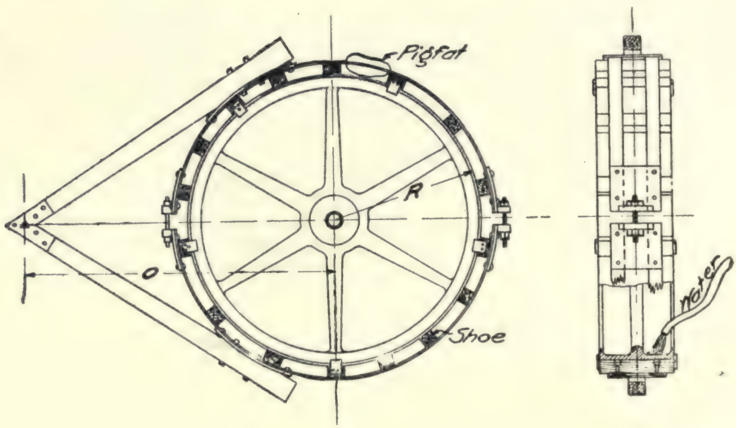


Fig. 370.

tested at Purdue University. The wood shoes are placed two or three inches apart and are fitted to the face of the pulley. Wrought iron straps press these against the face.

The flanges shown in the part sectional view are intended to hold water which is poured in for cooling purposes. The Westinghouse Company have found that for oiling the break shoes " green " pig fat is the best. A large slice is laid upon the top of the shoes and the heat allowed to melt it.

To get the proper area for brake shoes, the author has the following formula: $A = \frac{P}{.0002 \times S \times R}$, wherein A is the total

area of the shoes in square inches; P the power in horse power to be dissipated; S the speed in revolutions per min., and R the radius, in feet, of the pulley.

This is derived from dynamometers actually made and used by various engineers. About one-half to one-fourth of the area of the pulley face should be covered by the shoes.

For brakes not cooled by water use $A = \dfrac{P}{.00008 \times S \times R}$

The capacity P of this dynamometer in horse power (the principle is the same for other types) is found from the following formula:

$$\frac{1.001 \times A \times p \times R \times \text{r.p.m.}}{33{,}000} = P,$$

where A = the area in square inches of the frictional surfaces of all the shoes; p = the assumed pressure per square inch of the shoe on the rim of the wheel, and may be taken at about three pounds. 1.001 is a constant found by multiplying the coefficient of friction, .35, by π; R is the radius of the wheel in feet.

This formula must not be confused with that for the measurement of the power as in that case R is the radius at which W is applied and $= O$, Fig. 370. It is only used to get at the proper size of the brake.

For testing large turbines a less expensive, and at the same time a very satisfactory way is to test the electrical generators driven by them. Of course this can only be done when there is direct connection as the efficiency of a line of shafting would otherwise have to be found.

Hydro-Compressors.

No book on hydraulics would be complete without something on hydro-compressors, a method of power development that is to become quite common in the future.

Fig. 371 shows in its entirety such a compressor. The letters show the important parts as follows: A, the penstock; B, the receiver; C, the compressor pipe; D, the air chamber or collector; E and F, the tail race; G, timbering used, where shaft is sunk in earth, to support the walls; H, the blow-off pipe; I, the compressed air-feed pipe; J, the air head consisting of; a, the tel-

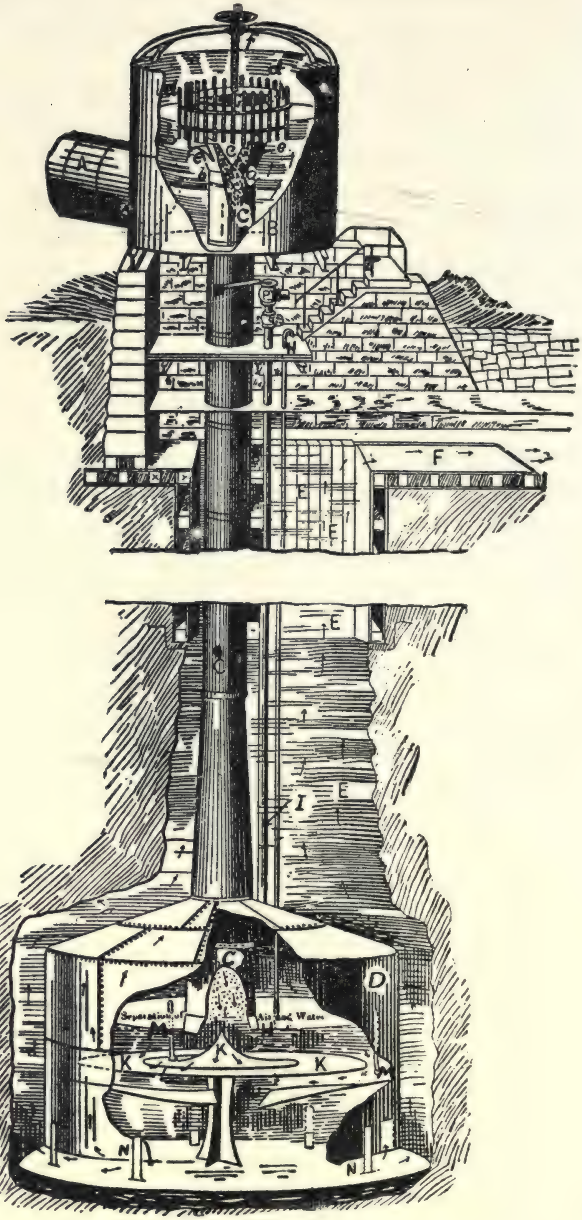


FIG. 371.—Hydro-compressor.

escoping pipe with bell-mouthed casting, b, opening upwards; c, the cylindrical and conical casting; d, the vertical air supply pipes, each having at its lower end a number of smaller inlet pipes radiating from it towards the center of compressor pipe; e, the adjusting screws for raising the air-head; K, the diffuser; L, the apron; M, the pipes to allow the escape of air from beneath apron and dispenser; N, the legs by which the separating tank is raised above the bottom of the shaft to allow egress of the water.; P, the automatic regulating valve.

The water is conveyed to the tank B through the penstock A, where it rises to the same level as the source of supply. In order to start the compressor the head piece J must be lowered by means of the hand-wheel f so that the water may be admitted between the two castings b and c. The supply of water to the compressor, and consequently the quantity of compressed air obtained, is governed by the depth to which the head piece is lowered into the water. The water enters the compressing pipe between the two castings b and c, passing among, and in the same direction as, the small air inlet pipes d. A partial vacuum is created by the water at the ends of these small pipes, and hence atmospheric pressure drives the air into the water in innumerable small bubbles, which are carried by the water down the compressing pipe C. During their downward course with the water the bubbles are compressed, the final pressure being proportional to the column of water sustained in the shaft E and tail race F.

When they reach the disperser K their motion is changed, along with that of the water, from the vertical to the horizontal. The disperser directs the mixed water and air towards the circumference of the separating tank D. Its direction is changed again towards the center by the apron L. From thence the water flows upward, and, free of air, passes under the lower edge of the separating tank. During this process of travel in the separating tank, which is slow compared with the motion in the compressing pipe C, the air, by its buoyancy, has been rising through the water and pipes M, M, from under the apron and disperser, to the top of the air chamber D, where it displaces the water. The air in the chamber is kept under a nearly uniform pressure by the weight of the return water in the shaft and tail race.

The air is conveyed through the main *I*, up the shaft to an automatic regulating valve, and from thence to the engines, etc. The air pressure in the main and air chamber increases one pound per square inch for each two feet three and a half inches that the water is displaced downwards in the air chamber by the accumulating air. The variation in pressure from this source will not be more than three pounds per square inch in a working plant. As the automatic valve requires a change of only one pound per square inch pressure to close it completely it will be evident that, by properly adjusting the valve, some air can always be retained in the air chamber, and that the water can be prevented from ever reaching the inlet to the air main.

If a large quantity of air has accumulated in the air chamber, the valve allows of its free passage along the main; but when the air is being used more quickly than it is accumulating, and the pressure decreases below a certain point because the chamber is nearly emptied of air, the valve shuts partially, or completely, adjusting itself to the supply from the compressor. When the air has displaced the water almost to the lower end of the compressing pipe, it escapes through blow-off pipe *H*. A hydro-compressor was built at Magog, P. Q., and tested by a number of experts and was found to have an efficiency in relation to the power of the falling water of from 55 to 71 per cent. An old steam engine driven with the compressed air gave 51.2 h.p. for each 100 h.p. in the falling water. When the compressed air was heated to 267 degrees F., the efficiency was 61.5 per cent. If this heated compressed air had been used in a modern hot air jacket engine the efficiency would have been 87½ per cent.

Another compressor at Ainsworth, B. C., gave 71 per cent. efficiency. Much depends on the number of the air pipes and the air-head should be so made that pipes may be added or taken out. The number should be greatest when about half the water is used and reduced at full flow.

A new and very large compressor plant is that located at Victoria Mines in Michigan. The author is indebted to Mr. W. O. Webber for the following data:

The minimum quantity of water available is 29,000 cubic feet a minute. The power available is 4000 h.p.; the dam is 28

feet in height, and is a mile above the site of the compressor, and the extreme height from the top of the dam to the outlet of the compressor is 72 feet. There are three downflow pipes in solid rock, 5 feet in diameter, and 334 feet deep; they are lined with 6 inches of concrete. The air chamber at the bottom has a capacity of 82,000 cubic feet of air. The surface pipe is 24 feet in diameter, and the blow-off pipe 12 inches in diameter. Each one of the downflow shafts can be operated independently, and furnishes an equivalent of 1300 h.p. The air is furnished at a pressure of 118 pounds per square inch. The total cost of compressor, including dam, was $200,000.

The water is received into the downflow shafts over a circular shaped apron five feet in diameter. The apron is of steel

TABLE XLI

DATA FROM EXISTING HYDRO-COMPRESSORS.

H.P. developed.	Diam of. tube, inches.	Diam. of shaft, feet.	Gauge press, pounds.	Head, feet.	Depth of shaft, feet.	Location of Plant.
587	33	7.	46	107	210	Ainsworth, B. C.
300	18	3.5	25	14	64	Petersborough, Canada.
150	44	7.	52	16	150	Magog, P. Q.
1500	168	24	91	18.5	240	Norwich, Conn.
4000	60†	60	118	72	334	Victoria Mines, Mich.

† There are three of these 5-foot tubes.

construction, and weighs 11 tons. Five iron pipes, six inches in diameter, extend above the water to a distance of four feet, and connect several inches below the water line with 2000 small pipes, 0.25 inches in diameter, each small pipe pointing toward the center of the apron. The small pipes are about 18 inches in length, and, being arranged in a circle, there remains a space in the center of the shaft 3.5 feet in diameter.

As the water flows into the shaft, air is drawn down through the larger pipes, and is forced into the water as it passes over the ends of the small pipes. The separating chamber and separator consists of expanding tubes downwardly projecting into the air chamber, with conical concrete diffusers formed on the floor of the separator chamber below them.

It must not be supposed that the compressor pipe has in all

cases to be placed in a well. It may take the form shown in Fig. 372.

Frizell obtained an efficiency of 52 per cent. under a 5-foot head. The compressor is especially adapted to low heads.

In designing a compressor the head and quantity of water being given, first decide on the pressure for the air. Of course the higher the pressure the cheaper the engines and pipe line, but the efficiency of the compressor is less for high pressures. The loss due to absorption of air by the water averages about four per cent. and varies as the square of the pressure.

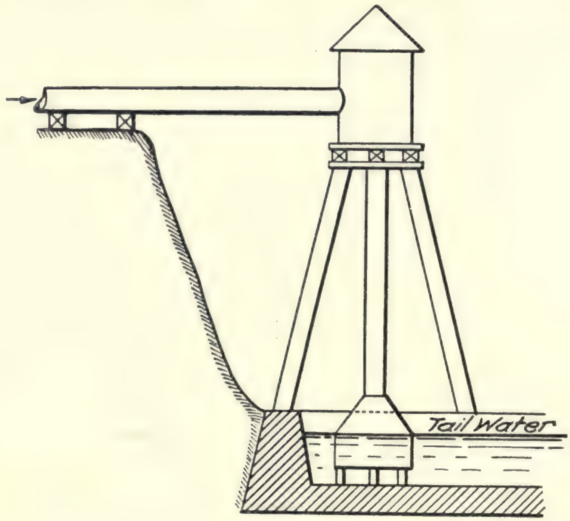


FIG. 372.

About 80 pounds pressure per square inch should be attained, though the costs of all the various machines will have to be the determining factor. Having decided on the pressure, find the length of the compressor pipe C, by multiplying the pressure by 2.3, which gives the length in feet. The diameter of this pipe depends on the volume of the water and the amount of head which can be lost. Its diameter is figured in the same way as that of a penstock. See pages 26 and 199. The effective head acting on this pipe is that due to the difference between the head and tail water levels, minus the head lost by the water ascending the shaft. The velocity in the shaft E should not exceed four feet per second.

The diameter of the air reservoir or collector is more a matter of judgment, but its area may be from 10 to 15 times that of the compressor pipe.

The air mixing pipes *d* may be of gas pipe of from $\frac{3}{4}$-inches to two inches in diameter and having radial pipes as shown in Fig. 373. The holes in the radial pipes are on the underside so that the water falling about them sucks the air down and out of the pipes.

Fig. 374 shows the Norwich compressor. The air head is mounted on a pipe which telescopes into the compressor pipe allowing an up and down movement equal to the variation of the level of head water. Where the fluctuations are more than a foot or so the level should be controlled at the head gates. The same head gates and racks suitable for a turbine plant are used

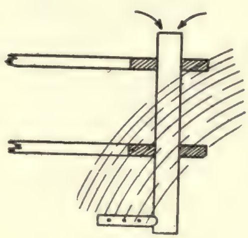


FIG. 373.

for a hydro-compressor, the rack, however, having a fine brass wire screen to catch every particle of drift. That part of the steel work containing the compressed air must be air-tight. The receiver must be protected from the cold in the northern climates as the air inlet pipes will freeze solid when the compressor shuts down.

Where the water used exceeds, say six to ten thousand cubic feet per minute the plant should be divided up into units, there being a common shaft and air chamber, but separate compressor pipes, receivers, diffusers and aprons. To avoid obstructing the flow as much as possible the author would suggest as arrangement of inlet pipes as shown in Fig. 375, the pipes being open at both ends and also having small holes drilled at the lower ends. The curve in the pipes corresponds to the flow of the falling water, so that the water runs along instead of against them.

By passing the compressed air through a heater and raising it to about 300 degrees Fahrenheit, 50 per cent. more power is obtained. To do this, according to Mr. Wm. Webber, requires

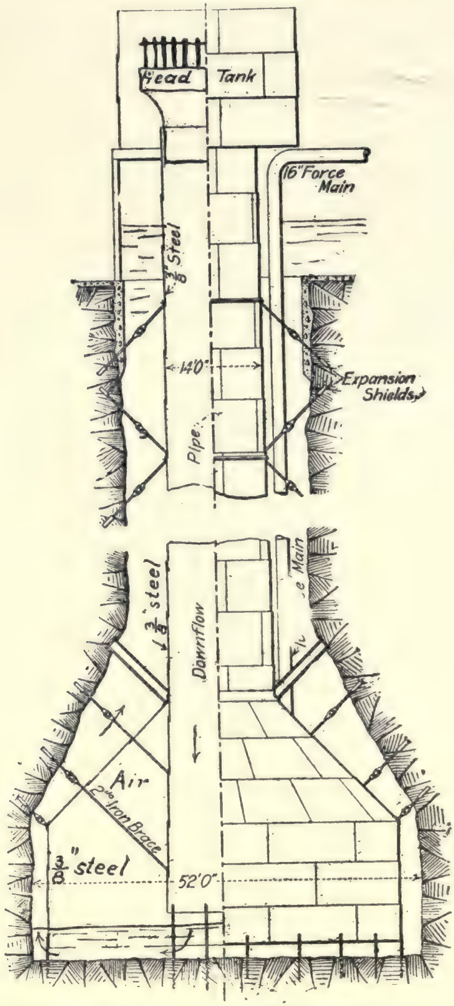


Fig. 374.

about $7\frac{1}{2}$ per cent. of the power in the river, figured on the basis of the amount of coal used.

Moistening the dry compressed air in the engine cylinder also

adds to the power. To saturate the air, the water has to be forced into the engine cylinder against the air pressure. Each h.p. in the river requires about 3.7 pounds per minute of water for this saturation, and the work performed by the pumps is found by $\frac{(P \times W \times 3.7)^2}{33,000}$ = power required, where $P = \frac{\text{Pressure of air in Receiver}}{.434}$ and W is the theoretical horse-power of river. The minuend is multiplied by two as the efficiency of a pump is about 50 per cent. The power thus found is about $4\frac{1}{2}$ per cent. of the river power.

It only requires a glance at a compressor to see that as compared with a turbine plant it is simplicity itself. There are no moving parts to wear out. No expensive and troublesome governors are required. No attendance. No oil or wear. No

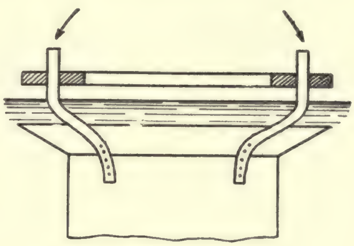


Fig. 375.

fire insurance, and practically no depreciation. Everything is automatic and it can be built under any and all conditions of foundation.

Professor Unwin states that it is practical to transmit power by compressed air to a distance of 20 miles with a loss of 12 per cent.

The cost is less than a turbine plant, as there are no journals, shafting, gearing, etc., the only cost being for the boiler iron and excavation. The cost of excavation will of course vary with the condition. Rock excavation will cost \$5 to \$8 per cubic yard and earth from 50 cents to \$3.

Mr. Webber places the cost of a 5,000 h.p. compressor at \$42,000. The boiler iron ought not to cost more than four cents per pound erected. Of course the same dam, head gates, canals

and racks are required as for a turbine plant, but no power house.

For distances less than five or six miles (and no doubt the future will see this increased), the transmission of the power by means of compressed air is as efficient as by any other means, a two per cent. loss being usually allowed. A velocity of 60 feet per second, may be allowed in the pipes, and as each horse-power at 85 pounds pressure takes about 1.44 cubic feet of air per minute, the area of the pipe may be determined. Webber gives the cost of a 20-inch steel pipe four miles long carrying 5,000 horse power at 85 pounds pressure as $3.05 per foot laid, making the cost per mile $18,500, and for four miles $74,000. An electric transmission would cost as follows:

Two governors (for two units)..........	$2,000
Generator house......................	5,000
Switch board.........................	2,000
Four miles transmission line...........	4,500
Generators and exciters...............	50,000
Step up and step down transformers....	30,000
	$93,000

The cost is more in favor of the hydro-compressor plant, as the distance grows less and vice versa.

Compressed air may be used in the engines already operating a factory, but the greatest efficiency is obtained when each machine is driven by its own air engine, in which case the efficiency is about that of the electric motor.

Water is sprayed into the cylinder of the air engine, but any engine may be fitted with a spray.

Small air motors of about ½ h.p. require as high as 14 cubic feet of air at 80 pounds pressure per square inch per minute per horse power.

Auxiliary Plants.

The same course of reasoning as applied to a storage battery plant, see page 421, should be pursued in selecting the proper size of the auxiliary plant, the chief difference being that the auxiliary plant is used to tide over the months of low water rather than the daily fluctuations of load, and the curves showing the monthly fluctuations of the river flow are used in determining the proper size of the plant rather than the hourly

variations in the power output. Usually the size of the auxiliary has to be guessed at, as accurate data on the river flow is seldom to be had.

STEAM PLANT.

Boilers.

In considering an auxiliary plant for a water power it must be borne in mind that ordinarily it will be in use but about one-third of the time. Therefore a boiler should be selected which will depreciate the least when not in use. The kind of boiler selected should next depend upon the price of coal.

Table XLII gives relative economy, etc., of the various types of boilers.

TABLE XLII.

Comparison of Various Types of Boilers.

Type of Boiler.	Sq. ft. of heating surface for 1 h.p.	Coal per sq. ft. Heating Surface per hour.	Relative economy.	Relative Rapidity of steaming.	Authority.
Water-tube........	10 to 12	.3	1.00	1.00	Isherwood.
Tubular...........	14 to 18	.25	.91	.50	Isherwood.
Flue..............	8 to 12	.4	.79	.25	Prof. Trowbridge
Plain Cylinder......	6 to 10	.5	.69	.20	
Locomotive........	12 to 16	.275	.85	.55	
Vertical Tubular....	15 to 20	25	.80	.60	

In actual practice, all day firing, and for small lighting plants having compound condensing engines, 10 pounds of coal is burned for each kw-hr at the switch board. In badly designed, small, isolated plants the consumption may reach 15 pounds per kw-hr.

The coal item is about half the entire cost of steam power. A locomotive boiler will evaporate (usual practice) from 6 to 8 pounds water per pound of coal, while a water tube boiler will evaporate 7½ to 9 pounds. Therefore, if we wish to install a 1000 h.p. boiler plant where fair coal costs $1 per ton (2240 pounds is always considered a ton of steam coal), and where the plant will be run 1000 hours per season of

drought, the value of the difference between the coal used by the water tube and the locomotive boilers is first determined.

TABLE XLIII.

FUEL AND WATER REQUIRED FOR THE PRODUCTION OF MECHANICAL ENERGY.

	Lbs. water from and at 212° per lb. of coal.	Lbs. coal per h.p. per hour.
	9	3.83
Good coal and boiler	10	3.45
Fair coal and boiler	8.6	4.
	8.	4.31
	7.	4.93
Poor coal and boiler	7.	5.
	6.	5.75
	5.	6.90
Lignite and poor boiler.	3.5	10.

From Table XLIII it is seen that for a first class boiler of the water tube type about 3.7 pounds, of coal will be used per h.p. per hour, and 1000 h.p. for 1000 hours will use

$$\frac{1,000,000 \times 3.7}{2240} = 1652 \text{ tons}$$

which, at $1, will cost $1,652. A locomotive boiler will consume about six pounds of fair coal per h.p.-hr., or in the above case $\frac{1,000,000 \times 6}{2240} = 2,680$ tons costing $2,680.

Table XLII also gives the relative efficiencies for the same coal. The water tube boilers will cost, set up, about $10,000, not counting buildings or smoke stacks, as they would cost about the same for all boilers, except where building sites are very expensive and fine buildings are erected. The locomotive boilers would cost about $7000. The difference in cost of operation is about $1000 per season in favor of the water tube boilers, and difference in cost of plant is as follows:

Difference in first cost.	$3,000
" " interest on investment.	180
" " maintenance.	300
Total.	$3,480

About 3½ to 4 years service would pay for a first class boiler plant if the coal cost $3. In one season's run, the water tube boilers would save $3000 over the cheaper boiler, an amount which just pays the difference in their cost.

Another strong argument for the water tube boiler is its freedom from disastrous explosions and ease of repair.

The author would strongly advise the intending purchaser to look up the second-hand boiler market. Frequently half the cost of a new boiler plant can be saved by the use of a second-hand boiler, which is practically as good as new. Of course great care must be taken in selecting such boilers. The Babcock & Wilcox and the Heine water tube boilers are among the best.

Table XLV and diagram will give a good idea of the dimensions of a boiler plant. The general proportions will hold good for any number of fire tube boilers. The setting shown is of brick, but it may readily be made of concrete, using the same dimensions. The setting is for tubular boilers; water tube boilers are taller, but will take up about the same floor space.

It is impossible to give the table of sizes for water tube boilers, as the various types vary so widely, but below is given the relative dimensions of three of the most prominent makes:

TABLE XLIV.

OUTSIDE DIMENSIONS OF WATER TUBE BOILERS.

Make and Nominal Rated Size of Boiler.	Width.	Length.	Height.
Babcock & Wilcox, 370 h.p.	13′— 8″	17′—9½″	19′— 6″
Babcock & Wilcox, 2 in battery	26′— 6″	17′—9½″	19′— 6″
Sterling, 500 h.p.	18′— 0″	19′— 0″	23′— 0″
Babcock & Wilcox	15′— 5″	23′— 5″	20′— 0″
Vertical Wickes, 400 h.p.	22′—11″	19′— 5″	34′— 0″

Roughly, each horse power requires about 0.666 square foot of floor space.

The volume occupied by water tube boilers is about as follows, for the largest sizes:

VOLUME OF WATER TUBE BOILERS PER H.P.

Babcox & Wilcox	13.75 cubic feet
Sterling	15.73 " "
Wickes	18.70 " "

TABLE XLV.

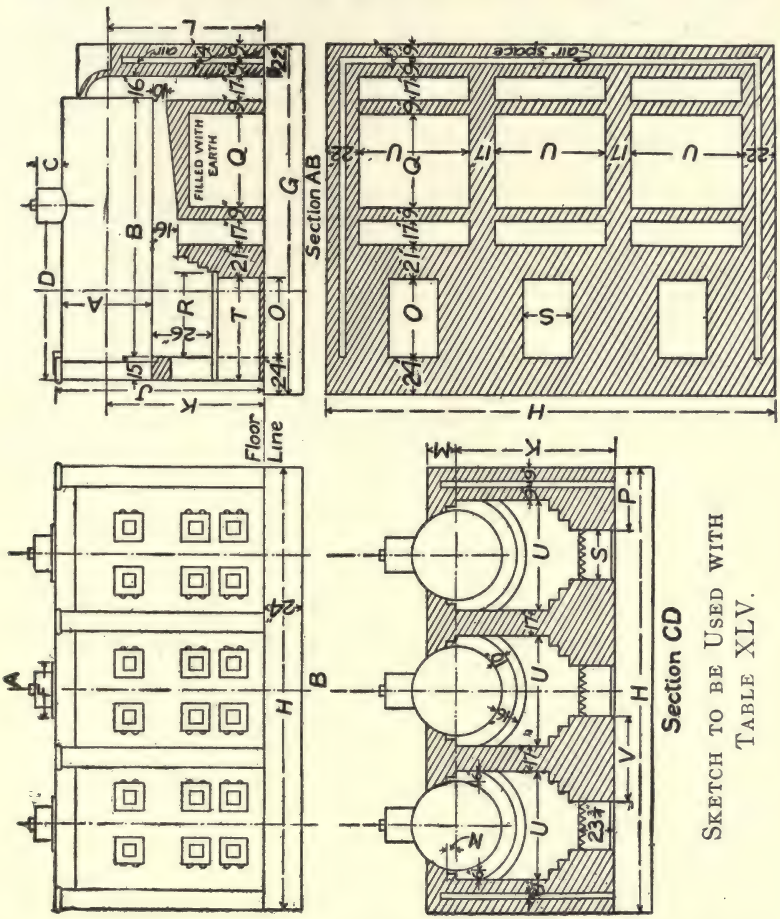
J. FEHRENBATCH, M.E.

FIRE TUBE BOILER SETTING.

Boilers.							Setting.											Furnace.					
Nominal rated h.p. of each boiler.	Diameter of boilers.	Length of boilers.	Top of boilers to top of Dome flange.	Center of independent dome to front of boiler.	Center of riveted dome to front of boilers.	Width of stack saddles.	Length of foundation.	Width of foundation.	Floor line to top of safety valve	Floor line to top of fire fronts.	Floor line to center of boilers, front end.	Floor line to center of boilers, back end.	Center of boilers to top of walls.	Center of boiler to closing in of side walls.	Front walls to bottom of bridge walls.	Thickness of outside jambs.	Length of aprons.	Length of furnaces.	Width of furnaces.	Length of ash pits.	Width of soot and back pits.	Thickness of middle jambs	No. of surface feet in grate's surface.
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	
	in.	ft.	in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	in.	in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	sq. ft.
55	52	14	27	8—7	7—6	3—9	19—0	22—6	12—3	9—2	6—4	6—3	14	9	4—1	2—6	5—0	4—2	4—0	5—6	5—4	2—9	49.98
60	54	14	27	8—7	7—6	3—9	19—0	23—0	12—5	10—2	6—5	6—4	13	8	4—1	2—5	5—0	4—2	4—4	5—6	5—6	2—7	54.15
70	54	16	27	9—10½	8—6	3—9	21—0	23—0	12—5	10—2	6—5	6—4	13	8	4—1	2—1	7—0	4—2	5—0	5—6	5—6	1—11	62.49
75	60	14	33	8—7	7—6	4—7	19—0	24—6	13—6	10—2	6—8	6—7	13	8	4—1	2—3	5—0	4—2	5—2	5—6	6—0	2—3	64.56
85	60	16	33	9—10½	8—6	4—7	21—0	24—6	13—6	10—2	6—8	6—7	17	12	4—1	2—2	7—0	4—2	5—4	5—6	6—0	2—1	66.66
100	66	16	33	9—10½	8—6	5—0	21—0	26—0	14—4	11—2	6—11	6—10	18	13	4—7	2—5	6—6	4—8	5—4	6—0	6—6	2—7	74.64
125	72	16	33	9—10½	8—6	5—6	21—0	27—6	14—7	11—2	7—2	7—1	16	11	4—10	2—4	6—3	4—11	6—0	6—3	7—0	2—5	88.50

The capacity of the boiler plant is found for all ordinary loads as follows:

The kilowatt capacity of generators times 0.75 equals horse power of the boilers. Nominal rating is used in above; of course, where a water power is to take all the peak loads this would



SKETCH TO BE USED WITH TABLE XLV.

give too much. Then one-half the kilowatt capacity of generators would be sufficient.

By heating the feed water with the exhaust about 6 to 10 per cent. of the fuel will be saved, besides saving the boilers from the evil effects of filling with cold water.

If slack coal is used it should be blown into the furnace.

The boiler house should be separated from all other rooms by a thin wall.

In most cases steel smoke stacks will be found the most economical for an auxiliary plant. There should be at least two for plants of any size, and for large plants about one stack per 1000 h.p. One stack for 1000 h.p. will cost, all set up, about $1,500, and for 500 h.p., $500. A Weber reinforced concrete chimney, 10 feet inside diameter, and 200 feet high, with foundation, will cost about $7,000.

TABLE XLVI (Kent).

SIZE OF CHIMNEYS AND PROPER H.P. BOILER.

Diam. of chimney inches	Height of Chimney in Feet.								
	50	60	70	80	90	100	110	125	150
	Commercial H.P.								
18	23	25	27						
21	35	38	41						
24	49	54	58	62					
27	65	72	78	83					
30	84	92	100	107	113				
33		115	125	133	141				
36		141	152	163	173	182			
39			183	196	208	219			
42			216	231	245	258	271		
48				311	330	348	365	389	
54				363	427	449	472	503	551
60				505	539	565	593	632	692
66					658	694	728	776	849
72					792	835	876	934	1023
78						995	1038	1107	1212
84						1163	1214	1294	1418
90						1344	1415	1496	1639
96						1537	1616	1720	1876

The quality of the coal has much to do with the size of the chimney. The height should be about 75 feet for free burning bituminous coal, 115 feet for slow burning bituminous coal or slack, and 125 to 150 feet for anthracite.

The steel stack rests on a concrete or masonry foundation and should be bolted firmly to it. Galvanized iron cable is used to guy the stack and these and the anchorage should be designed to withstand a wind pressure of 25 pounds per square foot against the stack. Suppose we have the case shown in

Fig. 377. The guys, supposing there are four, will support 68 lineal feet of the pipe against the wind pressure or a pressure of $5 \times 60 \times 25 = 7{,}500$ pounds.

If the guy is at an angle of 45 degrees the tension will be $7{,}500 \times 2$ since the tension is proportional to the distance $\frac{D\,C}{B\,C}$, therefore the cable will have to sustain 15,000 pounds (see Table LXIII). This is more than would be ordinarily given to one guy, as another set would be attached further down, but the above will serve to caution the builder against the usual practice of using any and all kinds of anchorage and cables, in the blind hope that no great storm will strike the stack.

A good boiler should, under forced firing, be capable of evapo-

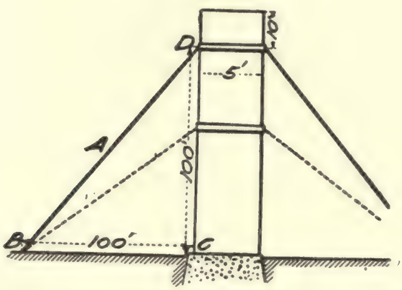


Fig. 377.

rating twice normal. Therefore in selecting the proper capacity of the boiler plant take the peak load, as found on page 426, and make the boiler capacity one-half the peak. Boilers should be operated 25 per cent. above the rated capacity to give best results.

It often occurs in the case of a water plant, that, while there is not enough water to carry the average load there is enough, using a storage reservoir, to carry the peak loads.

In this case the water is all reserved for the peaks and the boilers run steadily and efficiently on the average load. In which case the capacity of the boilers is made equal to three-fourths the *average* load.

In a large plant there should always be at least one spare boiler. The feeding should not be left to an injector alone, but both an injector and a pump should be used in all cases.

Steam Engines.

The engine best fitted for auxiliary work is, undoubtedly, the high-speed automatic cut-off engine, though, as in the case of boilers, the price of coal has much to do with the selection.

TABLE XLVII.

COMPARATIVE EFFICIENCY OF ENGINES.

Kind of Engine.	H.P.	Steam Press.	Water per I.H.P., non-condensing.	Per hour, condensing
Plain slide valve with long stroke, cut-off $\frac{5}{8}$	25 to 100	75 to 80	40 to 50	30 to 40
Automatic, high speed, single valve, cut-off $\frac{1}{4}$	50 to 150	75 to 80	25 to 35	20 to 25
Automatic, four valve, and Corliss, high speed, cut-off 1/5	50 to 500	110 to 120	22 to 30	16 to 24
Compound automatic, four valve and Corliss, high speed	400 and up	110 to 120	20 to 27	13 to 20
Triple expansion	500 and up	120 to 160	20 to 27	$12\frac{3}{4}$ to 18

The true criterion of the engine's efficiency is the amount of water used per i.h.p.

By use of the above table in connection with Table XLIII it can be easily figured what the coal bill will be with the different engines. Of course the less efficient engine will require more boiler capacity, and this must be allowed for.

The high-speed engine governs to within 2 per cent from normal load to a sudden no load. Where space is valuable they are the thing, especially those of the vertical type, such as the Westinghouse. When not compounded they consume about 35 pounds of water per indicated h.p. per hr.

Roughly, a high-speed automatic cut-off engine will cost $14 to $17 per h.p., all set up. A Corliss cross compound slow-speed high-pressure engine will cost all set up about $30 to $40.

Usually second-hand engines may be purchased at half price, which will answer every purpose.

As the engine's power is given by the formula

$$H.P. = \frac{2PLAN}{33{,}000}$$

where P is the mean effective steam pressure in pounds per square inch on piston, A the area of the piston in square inches, N the

number revolutions per minute, and L length of the stroke in feet, it is evident that by varying the cut-off and therefore P, the power of the engine can be varied.

All engines are designed for a certain mean effective pressure, at which they are most efficient, and in selecting the proper size its capacity should just equal the average load, *unless* the permissible change in cut-off for the particular engine will not increase the power sufficiently to take care of the peak load. Suppose the case of a 100 h.p. engine cutting off at $\frac{1}{4}$ stroke, boiler pressure at 100 pounds; r.p.m. = 250, length of stroke $L = 1$ foot, and the area of the cylinder = 113 square inches.

Then at $\frac{1}{4}$ cut-off $\frac{2PLAN}{33{,}000} = 102$ h.p. And at $\frac{3}{4}$ cut-off

$$\frac{2PLAN}{33{,}000} = 165 \text{ h.p.}$$

Boiler pressure multiplied by C taken from table XLVIII is substituted in the above for P in each case. By changing the cut-off from $\frac{1}{4}$ to $\frac{3}{4}$ we add about 50 per cent to the engine's capacity. While this is at the expense of efficiency, it will be good practice to make 165 h.p. the peak load capacity of the engine. Even $\frac{7}{8}$ cut-off is advisable in this case. Engines having heavy flywheels can carry a 100 per cent. overload for a few seconds. As in the case of boilers, where possible, let the water power take care of the peak loads and keep the engines working as nearly as possible at full load and efficient cut-off.

TABLE XLVIII.

MEAN EFFECTIVE PRESSURE FOR DIFFERENT CUT-OFFS.

Point of cut-off	C	Point of cut-off	C
$\frac{1}{4}$	.5965	$\frac{5}{8}$	.9188
$\frac{1}{3}$	.6995	$\frac{2}{3}$	.9370
$\frac{3}{8}$	.7428	$\frac{3}{4}$	.9657
$\frac{1}{2}$	.8465	$\frac{7}{8}$	.9917

Boiler pressure $\times C$ = *Mean Effective Pressure.*

Condensing engines should not ordinarily be used for auxiliary work.

The common single valve engines have a very limited range of cut-off and cannot be depended upon to carry a prolonged overload of more than 25 per cent. If the engine carries a heavy fly-wheel a momentary over load of from 50 to 75 per cent. may be carried.

On the other hand the Corliss and other four valve engines have a very large range of cut-off $(0-\frac{7}{8})$, and will safely carry a momentary overload of 100 per cent.

THE INTERNAL COMBUSTION OR GAS ENGINE.

" The gas engine has probably developed more slowly than any other piece of modern apparatus, as it is now 30 years since the Otto gas engine was introduced. It is only within the last ten years that the larger type of engine, from 500 to 2,000 h.p. in size, has appeared. The delay in bringing forward the most efficient motive power known is chiefly due to the difficulty experienced in developing an efficient and inexpensive method of making gas. As far as the production of gas from anthracite and non-caking bituminous coal is concerned this problem has apparently been solved, but it is still in a more or less unsolved condition for the richer bituminous and semi-bituminous caking coals of the Eastern States.

" The following heat balance is believed to represent the best results obtained in Europe and the United States up to date in the formation and utilization of producer gas

" Analysis of the average losses in the conversion of one pound of coal containing 12,500 B.t.u. into electricity:

	B.t.u.	%
1. Loss in gas producer and auxiliaries.........	2,500	20
2. Loss in cooling water in jackets............	2,375	19
3. Loss in exhaust gases.....................	3,750	30
4. Loss in engine friction.....................	813	6.5
5. Loss in electric generator..................	62	0.5
6. Total losses..............................	9,500	76.0
7. Converted into electrical energy.............	3,000	24.0
	12,500	100.0

" The great objection to the use of the gas engine for electrical purposes has been: First, its lack of uniform angular velocity;

secondly, its uncertainty in action and high cost of maintenance; and thirdly, its inability to carry heavy overloads. Recent developments have removed the first and second objections; and a period of vigorous development has resulted in placing the gas engine in the front rank of claimants for attention as a prime mover.

"The total investment for a gas-producer plant, all auxiliaries, gas engines and electric generators, has been reduced by the elimination of the gas-holding tank to a point where it is now practically on a par with a first-class steam plant using high-grade reciprocating engines.

"Where natural gas or blast-furnace gas can be obtained, the gas engine has outdistanced all competitors; and now that some of our large manufacturers have taken up in earnest the problem of designing producer-gas plants, it is safe to say that rapid developments will result.

"The records of operation of several important installations of gas engines in power plants abroad and in this country seem to indicate that only one important objection can be raised to this prime mover, and that is that its range of economical load is practically limited to between 50 per cent. load and full load. This lack of overload capacity is probably a fatal defect for the ordinary power plant, more especially for the average railroad plant operating under a violently fluctuating load, unless protected by a storage-battery of comparatively large capacity.

"Over a year ago, while watching the effect of putting a large steam turbine having a sensitive governor in multiple with reciprocating engine-driven units having sluggish governors, it occurred to the author that here was the solution of the gas-engine problem; for the turbine immediately proceeded to act like an ideal storage-battery; that is, a storage-battery whose potential will not fall at the moment of taking up load, for all the load fluctuations of the plant were taken up by the steam turbine, and the reciprocating units went on carrying almost constant load, while the turbine load fluctuated between 0 and 8,000 kw. in periods of less than 10 seconds.

"The combination of gas engines and steam turbines in a single plant offers possibilities of improved efficiency while at the same time removing the only valid objection to the gas engine.

"A steam-turbine unit can easily be designed to take care of

100 per cent. overload for a few seconds; and as the load fluctuations in any plant will probably not average more than 25 per cent. with a maximum of 50 per cent. for a few seconds, it would seem that if a plant were designed to operate normally with 50 per cent. of its capacity in gas engines and 50 per cent. in steam turbines, any fluctuations of load likely to arise in practice could be taken care of.

"We have seen that the thermal losses in the gas-engine jacket-water amounted to approximately 19 per cent., and as the water is discharged at a temperature above 100° it can be used to advantage for boiler feed

"The jacket-water necessary for an internal combustion engine will probably be about 40 pounds per kilowatt-hour, assuming that the jacket-water enters at 50° F.; then the discharge temperature will be $50 + \frac{19 \times 12{,}500}{40 \times 100} = 109.4°$ F."

The above is quoted from Mr. H. G. Stott, and is one of the most authoritative and important discussions of the gas engine subject ever given, and marks the latest word in that branch of engineering. Mr. Stott is in charge of the largest steam power plant in the world, and being a man of great integrity and ability, he stands in a prominent position to treat the subject in a classic manner.

Therefore, from what he says the gas engine producer plant is, by far, the best auxiliary to install in connection with a water power having large storage capacity. The water power would take the fluctuating loads, leaving the steady average load for the gas engines.

In some cases the jacket-water could be used for heating, though, as a rule, this would be lost.

The gas engine is without doubt the ideal auxiliary power. There are no boiler plants to depreciate. The engine is ready at all times to take up its load. The modern gas engine is easily started, has good regulation and is as easily operated as the steam engine. There are many new makes on the market, but it is in the nature of an experiment when any but the standard engine is used. The Westinghouse, Otto & Priestman, Crosley, Koerting Cockrell and Snow are among the best.

The gas used may be a mixture of gasoline and air, lighting

gas taken from the city mains or producer gas. Using gasoline, the average engine will consume $\frac{1}{8}$ gallon of gasoline per effective h.p-hr. Using city gas, the consumption will be 21 to 22 cubic feet of gas per effective h.p.-hr. Using producer gas an engine should develop one i.h.p. per hr. with from 1 to 2 pounds good coal.

For plants of all sizes a producer plant should be installed. If the producer gas is made directly from the coal, and as indicated by the above the saving in coal is enormous The price of coal here again enters as a factor. A gas engine producer plant all complete with producer set up and ready for operation, would cost about $35 per brake h.p. using soft coal, wood, etc., and about $16 for a producer using hard coal, coke, etc. These figures do not include the engine.

Producer gas may be made from cheap bituminous coals, anthracite and coke. Usually small anthracite coal or coke is used, but bituminous coal, lignites and wood may be employed.

Tests show that a 16 h.p. producer plant gave one horse power for each 1.1 pounds of coal, and plants above 50 h.p. gave 1 h.p. for each $\frac{7}{8}$ pounds of coal.

Gas engines without producers cost at the factory as follows: Small engines of from 10 to 30 h.p. about $45 per h.p. Engines up to 100 h.p. $40 per h.p., and above that about $35 per h.p.

The complete plant will cost about $85 per b.h.p.

Electric Generators.

The type of generator depends upon the character of the load and the system of distribution. Therefore the various systems of distribution will de described in connection with the generators.

SYSTEMS OF DISTRIBUTION.

Continuous Current.

There are two general systems of continuous current distribution: the constant current and the constant potential systems. The constant current or series system is seldom used in this country except for series arc lighting. The lamps take about 10 amperes at about 50 volts per lamp. The generator must have a voltage equal to 50 times the number of lamps in series and a current capacity in amperes equal to 10 times the number of series circuits in parallel.

Knowing the number of arcs to be supplied for city lighting, and those for commercial lighting, the peak load can be decided on. Arc machines will carry 25 per cent. overload for a half hour, and their capacity need therefore only be 75 per cent. of the peak load.

The size being limited by $\frac{\text{(Voltage of the machine)}}{\text{Voltage of the lamp}}$ = to about 75 kw or less, the generator will be of the belted type.

It should rest on a heavy timber frame well oiled, and bolted with deeply countersunk bolts so as to well insulate it from the ground. The load is generally quite constant, and cheap governors may be employed for regulation.

The constant potential or parallel system is in almost universal use in this country. In this system the current capacity in amperes is equal to the current capacity of the lamp used times the number of lamps in multiple. There are three general methods of wiring, namely; the two-wire method for lamps and small motors (110 volts); the three-wire method, lamps and motors (220 to 440 volts) and the five-wire method for lamps and motors (440 volts).

Sharp peaks are sure to occur and the safe overload capacity of the generators must be at least three times the average load.

First class governing is an essential feature in the successful operation of this system.

The best practice is to have two machines, each being of sufficient size to carry the entire probable load as an overload of 25 per cent.

The machines may, and usually are, operated in parallel.

No precautions are necessary to insulate the frame from the ground.

Alternating Current.

In alternating systems there are two general classes, the single phase and the polyphase.

The single phase system may be high tension at the generator and stepped down at the load, or may be stepped up to very high tension at the generator and stepped down at the load.

The former has only one set of transformers, those for stepping down the voltage from 1000 to 5000 volts to 50, 100 or 200 volts. The secondary may be two or three-wire.

The latter has two sets of transformers, one for stepping up and one for stepping down. The transmission voltage may be from 5000 to 60,000 volts.

The advent of a single-phase motor has brought the single-phase system into great prominence, since, aside from the added cost in long transmissions, it possesses the following advantages

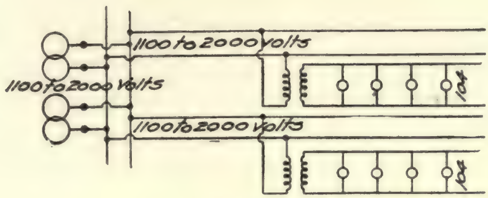


FIG. 378.

over the polyphase systems: By using single-phase a saving of 10 to 40 per cent. of the first cost of the motor transformer installation is affected; fewer transformers are required with a consequent saving in transmission losses of 10 to 20 per cent. Fewer meters are required, which is a good saving, as each small meter consumes about 1 per cent. of the power measured; also the labor

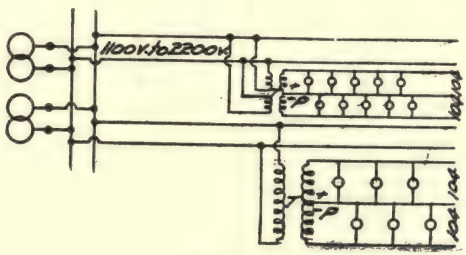


FIG. 379.

of installing each of the numerous meters is materially lessened; it costs less to erect the pole line, etc.

The above are very important facts and should be well considered.

The working voltage may be 100, 120, 200, or 240 volts.

One of the most common two-wire systems is that shown in Fig. 378, the two generators and the lamps being in parallel.

Two hundred and twenty-volt lamps are now used to some extent, in which case the distribution is materially cheapened.

In the three-wire system (Fig. 379) the transformers T are of large size, supplying a long line of lamps or small motors.

Either of the transformer connections shown may be used, care being taken to connect *unlike* terminals at P.

The polyphase system may be divided into the two-phase and the three-phase.

The two-phase, four-wire system, shown in Fig. 380, consists of two single-phase circuits differing in phase by 90°.

The chief advantage claimed for this system is that a rotating field may be established, which permits the use of two-phase

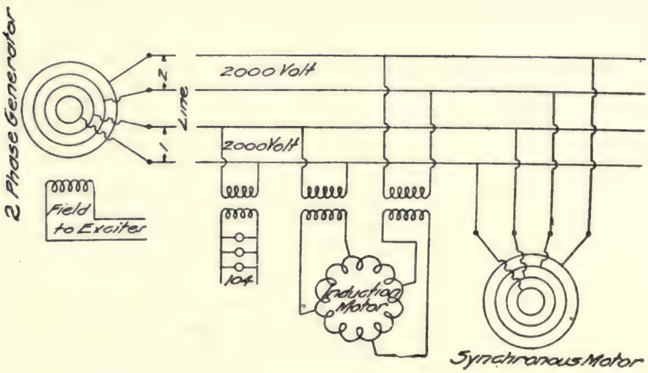


Fig. 380.

induction motors and the self-starting of synchronous motors. The two-phase induction motors will start up under full load, but the synchronous motors will not, being run up to speed unloaded. The success of the single-phase motor, of course, lessens these advantages so that the single-phase should be considered the better system up to 30 h.p.

Street railways have frequently been operated from this system synchronous converters supplying the continuous current. For long transmissions step up and step down transformers would be used.

This system requires the same amount of copper as the single-phase.

The three-phase system shown in Fig. 381 is by far the most

important alternating current system in use to-day. Especially is this true when considered in connection with hydro-electric power plants. By it power is being successfully transmitted at a pressure of 60,000 volts for distances as high as 150 miles. As shown above, no step up transformers are used, though, of course, for higher voltages than 10,000, and preferably voltages above 4300, the voltage is stepped up for transmission and then stepped down for use.

In comparing the actual operation of two-phase and three-phase systems there is little to choose between them. The three-phase saves 25 per cent. in copper over the single and two-phase systems on the transmission.

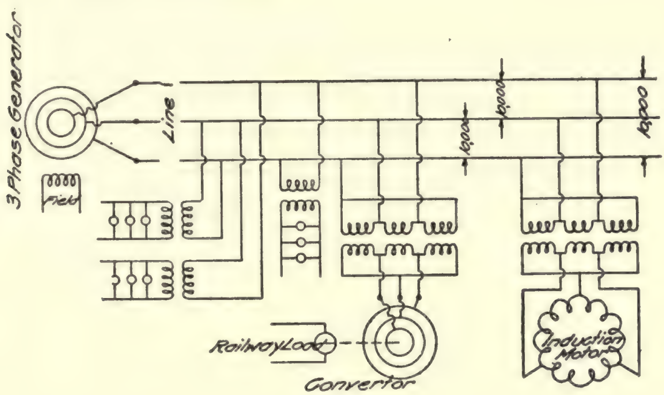


FIG. 381.

The selection of the proper size for the generators is one of the very important problems, and while an easy one, it is usually the cause for the most inexcusable mistakes.

At Constantine, Mich., there is a modern hydro-electric power plant built by a Chicago company. They have two 600 kw. generators having a maximum capacity (allowing for 50 per cent. overload,) of 2460 h.p. To drive these generators they have turbines with a maximum capacity, allowing for loss in gearing, of 1700 h.p.

This, too, is for the maximum working head of 11 feet. Frequently the head is reduced by back water to 6 feet, in which case they can hardly carry their small lighting load, though at the same time thousands of horse power are passing over the dam and going to waste.

This is not an extreme case, but one much better than is often found.

Generators are rated at a unity power factor. Therefore if the power factor is .80 the generator will have only about .80 of its rated capacity.

While electrical manufacturers do not guarantee it, yet it is a fact that any of the standard generators will carry a 25 per cent. overload right along and a 50 per cent, overload for two or three hours without doing them any injury.

Exciters for these generators should, in all cases, be belted to the shaft to give the most uniform velocity. Where the head fluctuates it is bad engineering to drive the exciter with one turbine, the speed of which cannot be controlled. The generator line shaft, when the plant is properly designed, is the best to drive the exciter from. In the case of horizontal turbines and direct connection it is very difficult to keep up the speed.

The latest method is to drive one exciter with a motor.

The loss of speed through diminished head reduces the voltage of the line current and also reduces the frequency. Where lamps are operated or where the hydraulic plant is operated in connection with some distant steam plant this reduction of the frequency is a very serious thing. The lamps refuse to work and the distant generators will not run in parallel.

A variation of 2 per cent. from the normal frequency may seriously affect the operation of the plant.

To get the proper size of generator, first find the greatest peak loads which are apt to occur. Divide this peak load by 1.5; this will give the rated capacity of the generator for a unity power factor. Now if the load is an inductive one and the power factor is .80 multiply the rated capacity as found from the above, by 1.2. This gives the commercial rating of the generator.

The exciter capacity should be about 30 per cent. greater for an .80 power factor than for a unity power factor. The exciter capacity for an .80 power factor should be about 4 per cent. of the generator capacity, as found from the above.

Switch Boards.

The switch board is the most variable factor in the design of the power house and one of the least understood. Everything

about the switch board should be thoroughly made and of the best materials. Marbelized slate for boards up to 600 volts is a good substitute for marble, the chief objection being that it scratches easily.

Whether of slate or marble care should be exercised in selecting slabs free from mineral veins. White Italian marble 1¼-inch to 2-inch thick is, in the long run, the most satisfactory. The Standard Marble Company of Cincinnati, O., sell a very good grade of marble. One and a quarter-inch costs about 65 cents, 1½-inch costs about 80 cents, and 1¾ to 2-inch costs about $1.10 per square foot.

The frame is in almost all cases made of angle irons as shown in Fig. 382, though especially prepared wood boiled in paraffin

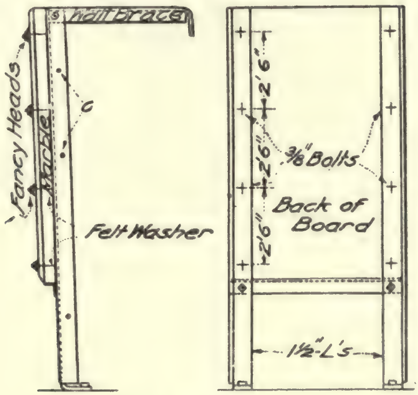


FIG. 382.

or linseed oil has been used for high tension work. A 1½ to 2-inch angle iron is plenty large for the ordinary board. Fancy heads are used on the face side of the board. The iron work must be painted before setting up. Felt washers are used between the iron and marble. The holes *c* are for fastening the various panels together.

There is no good reason why the engineer should not make the complete switch board on the site, and it is certainly a good plan for him to select the instruments with great care.

For eight or ten dollars a drill press, such as is used by blacksmiths, can be purchased with which to drill holes in the marble. It should be attached to a plank *A*, as shown in Fig.

383, and should be driven with a 1 h.p. motor The drill should have a speed of about 170 r.p.m., a table on which to lay the slab is very handy, though of course the floor (if perfectly level) will serve the purpose. It would take about one day to bore the holes for the board shown in Fig. 382.

The ordinary twist drill is used, plenty of water being fed to the cutting edge. Holes up to 7-16-inch may be drilled in one drilling; up to $\frac{7}{8}$-inch use two drills, and from that to $1\frac{1}{2}$-inch, use three different sizes. If the holes are not true file them with a rat-tail file.

To lay out the holes for drilling use an indelible pencil and prick punch. Where the drill has chipped the marble at back of

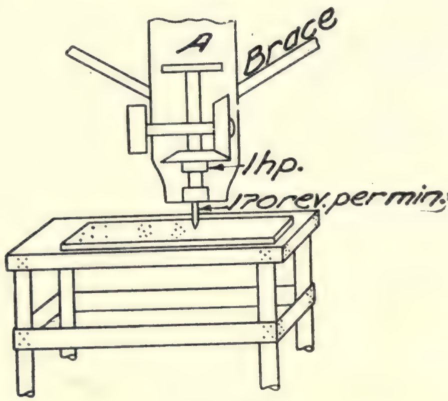


FIG. 383.

board fill out with plaster of Paris. When the individual panels are completed they are bolted together with $\frac{3}{8}$-inch bolts and temporarily stood up while the bus work is done. The back of a switch board is where the workman's skill is shown. In bus work use what is known as *half hard* copper bar. This may be had of the Detroit Copper & Brass Rolling Mills, Detroit, Mich. A bar thicker than 7-16-inch is very difficult to bend.

Aluminum makes excellent bus-bars, as it has a large area to dissipate heat and is light in weight. All connections to it have to be bolted.

One thousand amperes per square inch of copper and 750 amperes per square inch of aluminum is common practice for switches and bus-bars. All copper parts carrying currents of

opposite polarity must have a certain amount of air space between, as given in the table XLIX. This table may be used for either alternating or continuous current.

Due allowance must be made for the conditions at the board during operation, and if the atmosphere will be damp a wider arcing distance must be allowed.

TABLE XLIX.

SPACING OF BUS BARS AND SWITCH BLADES.

Voltage volts.	Current amperes.	Distance between nearest metal parts, inches.
0 to 125	0 to 10 10 to 25 25 to 50	0.75 1.00 1.25
125 to 250	0 to 10 10 to 35 35 to 100 100 to 300 300 to 1000	1.50 1.75 2.25 2.50 3.00
250 to 600	0 to 10 10 to 35 35 to 100	3.50 4.00 4.50
600 to 1000	0 to 10 100	5.00 7.00

Fig. 384 represents a simple bus-bar for boards of moderate size.

On very heavy bus-bar work instead of bending the bars, they are attached as shown in Fig. 385.

Care must be taken to maintain the proper arcing distance from the steel frame of the board. The flexible connecting

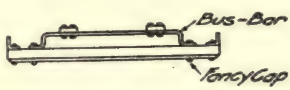


FIG. 384.

cables are *soldered* into the lugs as shown in Fig. 386 at *a*, but all strip connections as *c*, are bolted to the bus-bars with from one to four bolts.

Where the bus-bars are over $\frac{7}{16}$ inch thick it is well to make them of two or more strips with air spaces between for ventilation. A contact surface of one square inch per 100 amperes should be allowed at joints.

For heavy alternating currents the bus-bars may be made of tubing to keep down the losses due to skin effect.

In high tension work (6000 or more volts) the bus-bars are not usually placed on the board, but mounted on porcelain insulators behind it.

Where the boards are of such large size that a single attendant can not operate them the heavy switches are operated from a control board. Each switch is operated by an electric, pneumatic or hydraulic motor, which is put in motion from a small board representing on a small scale the large one. All the measuring instruments are mounted on the control board: fine wires connect them to the large bus-bars on the main board.

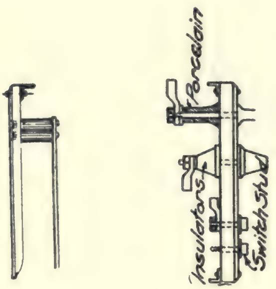


Fig. 385.

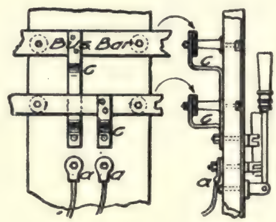


Fig. 386.

In an alternating current plant the current for operating the large switches is derived from a motor-generator in connection with a storage battery. Where the hydraulic head is sufficient, hydraulic pistons may be used, the valves being actuated by electro magnets.

Switches and Instruments.

Each switch is proportioned to the current and voltage. The higher pressures broaden and complicate them, while the heavy currents make the switch heavy and bulky. They may consist of one or more blades and are designated by letters which indicate the type of instrument as follows: S.P.S.T. means single pole (one blade), single throw (handle of switch in Fig. 386 does not open more than 90°). D.P.D.T. means double pole and double throw, as in Fig. 387, etc. Then the switches may or may not have fuses as in Fig. 388.

All contact surfaces must be so proportioned as to carry but 50 amperes per square inch of surface. These contacts should at full load not heat to more than 50 degrees Fahrenheit above the atmosphere.

A standard size for switch board panels is 62x30x2 inches for the main upper part, with a sub-base 28x30x2 inches.

In polyphase work, where there are several units, the usual practice is to have one panel for each generator and one panel for each exciter. The generator panel has the ammeter, volt meters, generator switches, fuses, field switch fuses, pilot lamp for generator, and on the back of the panel a lightning arrester for each phase, a station transformer and a ground detector.

Table XLIX gives the proper spacing distances for the metallic parts. Only in very small switches of high voltage should the hinge *a*, Fig. 388, be made to carry the current.

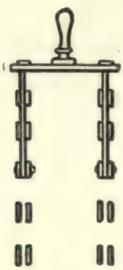


FIG. 387.

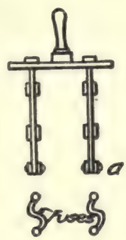


FIG. 388.

For switch board work the switches have lugs of sufficient length to project through the board and receive the strips, washer and nuts.

For higher tension switches of from 600 to 10,000 volts a barrier of marble, slate or glass is inserted between the poles or blades of the switch to prevent arcing, as in Fig. 389. This is really a single throw switch with one added contact making it a double throw suited for as high as 20,000 volts, though for tensions of from 6,000 to 20,000 volts an oil switch is considered the best.

Fig. 390 shows a high tension switch in which *A* is a copper piston and *C* a lever on the front of the board by which the eight pistons are brought into contact with the eight contacts *B*. The eight cylinders *D* are of porcelain and divided into pairs,

each pair having a connection for a wire from the generator and one to the line. The eight tubes therefore take care of four circuits.

Actual practice has demonstrated the oil switch to be the best, especially for inductive loads, and most compact for voltages up to 10,000, while for voltages above this a long break switch is considered the most reliable, dependance being placed on the length of break alone, which for 6,600 volts is 30 inches, for 22,000 volts 6 feet.

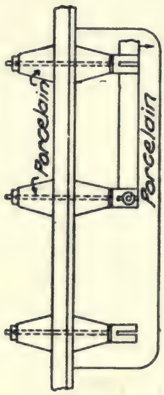


FIG. 389.

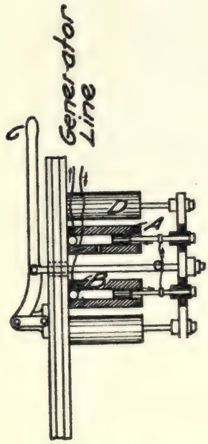


FIG. 390.

It is necessary to know the voltage of all machines connected to the board and also the voltage on the out-going feeders. The best, and what has become the standard voltmeter for direct currents is the Weston. It is mounted on the front of the board and a lamp provided to light the dial. All voltmeters must be dead beat and accurate to within 1 per cent. at all loads. It is quite essential to have at least two voltmeters so that one may be used as a check on the other, but it is not necessary to have one for each machine and feeder, as a multi-contact switch may be used to connect them with any circuit it is wished to get the voltage of. (See Fig. 391.)

For alternating currents it is customary to connect the voltmeters through a transformer, so as not to submit the instru-

ment to the high tension. These potential transformers are attached to the back of the board, or placed on the wall. They are very small transformers and should not carry any other load than the voltmeter.

Voltmeters are often mounted on swinging brackets at the ends of the board.

The range of all voltmeters should be about 50 per cent. above normal load.

For voltages of 10,000 and more an electrostatic voltmeter is often used. It may be connected in parallel across high tension lines without the use of a transformer.

Some voltmeters are so designed as to work on either alternating or continuous current.

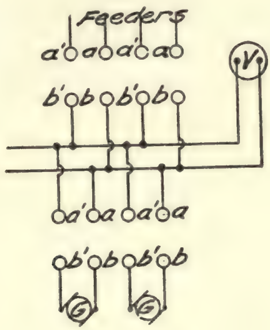


FIG. 391.

A voltmeter switch which is largely in use is shown in Fig. 391. By plugging in between a and b and a' b', any of the generators or feeders can be connected to the instrument.

One of the feeder lines in Fig. 391 could be replaced by pilot wires run back to the center of distribution. Then by connecting with the voltmeter the pressure at the far end of the line could be read. This method is only adapted for lines under two or three miles in length. A better way is to use a *compensator*.

This is a device by which the voltmeter reading is decreased by an amount equal to the drop in the line. The Westinghouse Mershon type is one of the best.

The connections for the Mershon compensator are given in

Fig. 392. A is an ordinary potential transformer, B is an inductance and C a non-inductive resistance, and D and E are small transformers. This is the most common arrangement and is suitable for the most inductive loads, such as motors or motors

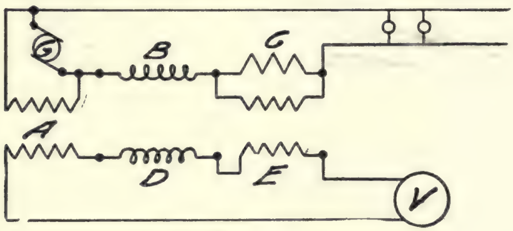


Fig. 392.

and lamps. For a small village lighting plant it is not thought necessary to use any such device as the telephone may be relied on to give warning of any dissatisfaction on the part ot the customers.

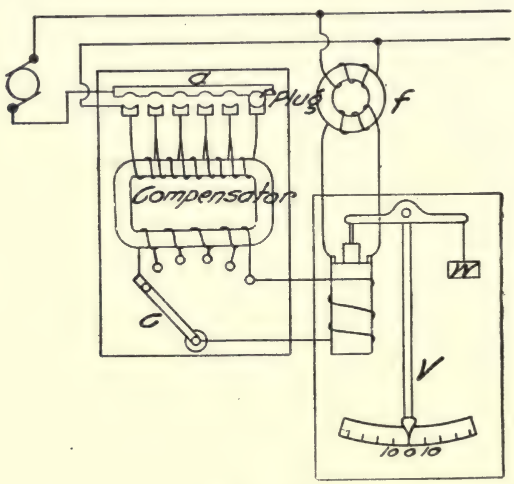


Fig. 393.

The Westinghouse Company manufactures a compensator suitable for lines having little self-induction, such as incandescent lighting. The connections are as in Fig. 393; f is the ordinary potential transformer. The voltmeter V is of the coil and

plunger type. When the voltage at the distributing end is correct the hand of the voltmeter is at 0. The adjustment is obtained by plugging in along *a*, and by rotating the contact *c*.

The Weston ammeter is the most widely used, whether direct or alternating, and is accurate to within 1 per cent. at all loads, and is also dead beat; that is, the oscillations soon cease after a change of load.

Usually, where the current is moderate, say less than 250 amperes, and the voltage not above 5,000, alternating current ammeters are connected directly in the main circuit, as in Fig. 394, but for high voltages and large currents a current transformer is connected as shown in Fig. 395.

A recording ammeter, whether for continuous or alternating current, should be part of every switch board equipment in order that the peak may be studied and due charges made. The Bristol Company of Waterbury, Conn., make both recording ammeters and voltmeters.

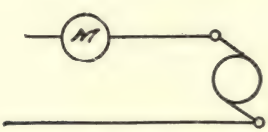


Fig. 394.

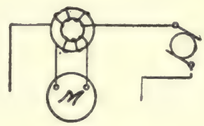


Fig. 395.

There should be an ammeter in each phase and one for the field exciting connections in addition to the watt-hour meters.

The circuit-breaker is the safety valve of a switchboard. Its purpose is to open the circuit at times of short circuit and overloads. It commonly consists of a solenoid which operates a trigger, releasing the switch when the current exceeds a certain predetermined amount. These instruments are quite expensive and may be considered as a luxury in view of the fact that the ordinary fuses can be relied on to a certain extent. However, it is an excellent plan to equip each generator and each feeder with a single-pole circuit-breaker.

Circuit-breakers serve to protect against lightning, though they should not be relied on for this purpose. They may also take the place of a switch. A great advantage possessed by circuit-breakers over fuses is the ease and quickness with which the broken circuit may again be put in operation. They are

also much more accurate than a fuse. The circuit-breakers should be so constructed that the main line breaker will act somewhat behind the branch line breaker, otherwise the main breaker may operate at the same time the branch opens, thus needlessly interrupting the service.

The contacts should be of carbon and the breaker should be able to carry a 75 per cent. overload. They may be had of as many poles as desired.

A good plan is to have a circuit-breaker on each side of the circuit, in which case one side will open automatically if it is attempted to close the other, while the short remains. If a single-pole breaker is used connecting the generator to the board it is best practice to place it on the negative side. There are circuit-breakers which open the circuit for both too high and too low currents. These are used in connecting generators to storage batteries.

Circuit-breakers are not quite so common on alternating current boards as they are on continuous current boards, but for high-tension transmission work they should certainly be used. When there is any uncertainty about the generators keeping in step, as in the case where turbines have no governors, it is advisable to connect them to the board through a breaker.

After years of experience with the Niagara transmission to Buffalo, a circuit-breaking switch has recently been installed on each phase of the outgoing feeders. Thus three three-phase transmission lines have nine breakers. The voltage is 22,000 and it was found that the break of the switch had to be fully six feet. The best type for high tension work is undoubtedly the oil circuit-breaker. In this type the break may be much shorter, as the oil quenches the arc.

Many boards have one oil circuit-breaker on each feeder panel, thereby protecting the board from overloads and shorts on the transmission line, but leaving the generators and exciters to the care of fuses.

Fuses form the most common method of providing an automatic interruption of the circuit during overloads and shorts. It is to their cheapness that they owe their popularity, for they are the most unsatisfactory part of a board's equipment. Depending on the fusing point of a metal, they are not at all accurate.

A common way is to have the fuses a part of the switch for low-tension work, but for high-tension boards it is advisable to have the fuses on the back of the board.

In Fig. 396 a General Electric fuse box is shown. The entire box may be pulled off the board, the slips *c* being only in frictional contact with the terminals *t*. These fuses are made for as high as 150 amperes and 2,500 volts. For higher tension boards the fuse blocks are placed at back of board and the blocks removed by means of an insulated pole some three or four feet in length to protect the operator from shocks.

All fuses should be enclosed, and a common form is enclosed in a fiber or hard rubber tube. On alternating current switch boards the practice seems to be to eliminate fuses as much as possible, though it is still common to place fuses on the feeders.

The wattmeter is to the power station what the journal and ledger are to the successful business man. In order to improve or

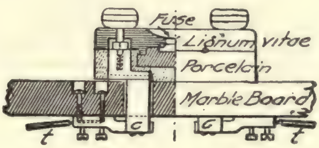


FIG. 396.

maintain the efficiency of the plant one should know the amount of power being sent out by each machine and by each feeder.

These instruments are usually mounted on the board, though if the board is crowded or for any other reason, they may be placed near the generator.

The power on a continuous current line may be obtained at any time by taking the product of the voltmeter and ammeter readings at the same instant, thus getting the power in watts, from which the horse power is obtained by dividing by 746 or the kilowatts by dividing by 1000.

The wattmeter, however, indicates this product so that the watts may be read off directly by simply pushing a button.

However, it is usually desirable to keep a record of the watt-hours, as the watt-hour is the unit on which the charge for power is based.

To get this continuous all-day record the watt-hour meter

is used. The best known instrument for this purpose is the Thomson, which may be used with either alternating or continuous current. In this instrument there are a number of dials at the top from which the total watt-hours may be read like the readings on a gas meter.

Fig. 397 shows a Thomson watt-hour meter connected to a two wire line.

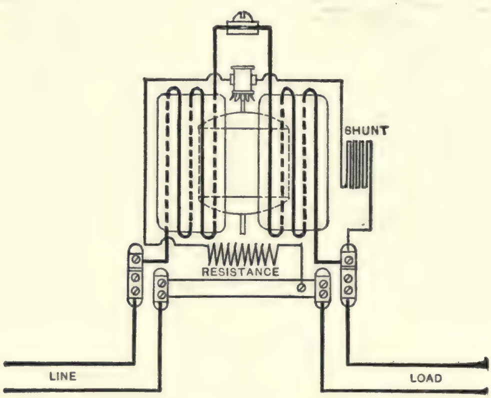


FIG. 397.

The power in alternating current circuits is equal to the product of the current and the in phase component of the e.m.f., and is obtained directly by connecting a wattmeter in each separate phase. In single-phase circuits the connections are precisely the same as in continuous current circuits. In un-

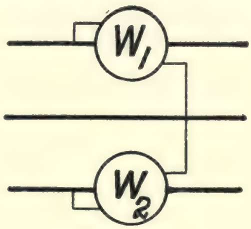


FIG. 398.

balanced two-phase circuits there should be a wattmeter in each phase. The total power equals the sum of the two readings, but in balanced two-phase circuits, such as motor load, there need be only one meter, and its reading, multiplied by two, will give the total power. In three-phase circuits only two meters are necessary, connected as shown in Fig. 398. The total power

is equal to the sum of the readings. The energy, which is equal to the product of the power and the time, is obtained with a watt-hour meter. For single-phase circuits the connections are the same as for continuous current circuits.

In two-phase circuits there should be two instruments if the load is unbalanced and one if balanced. The connection is the same as for single-phase, and the total energy is equal to the sum of the two readings, or twice the reading in one phase.

In three-phase, star-connected loads, one meter will measure one third the energy; in some cases the meter may be connected

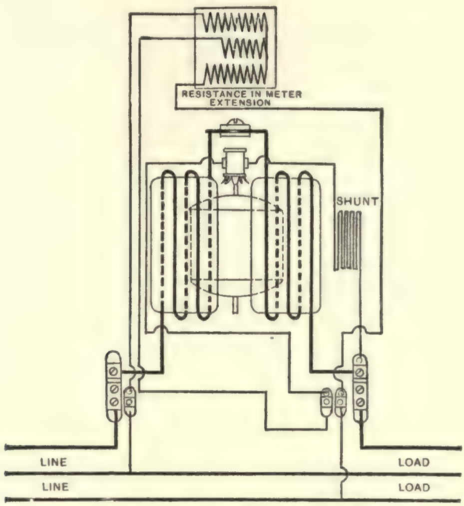


FIG. 399.

with an artificial neutral, as shown in Fig. 399. In unbalanced three-phase circuits two instruments connected as shown in Fig. 400 should be used.

Where the pressure is over 550 and under 3,000 volts it is not thought advisable to pass the line voltage directly through the instrument, so small transformers (*t*) are used, as in Fig. 401.

Induction watt-hour meters are sometimes used. These are for alternating currents only, and have to be adjusted to the particular frequency of the line. They, however, have no commutator to get out of order.

For series arc lighting the Thomson meter is connected as

in Fig. 402, a cut out, *a*, being provided to short circuit the line in case of an open circuit beyond.

In the case of very heavy currents a special meter is used, shown in Fig. 403.

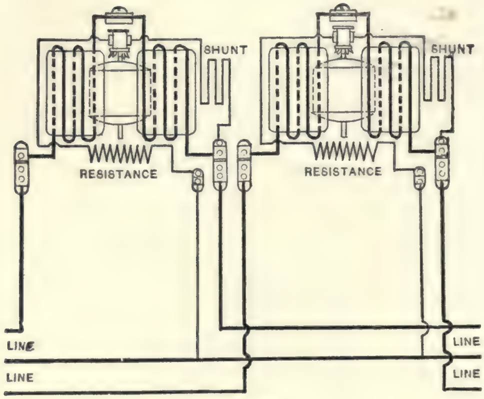


Fig. 400.

All meters should be accurate to within 3 per cent. They should maintain this accuracy throughout the load and be of sufficient size to carry the peak loads.

They should not waste more than 1 per cent. of the energy delivered to them.

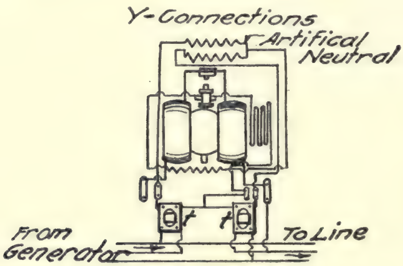


Fig. 401.

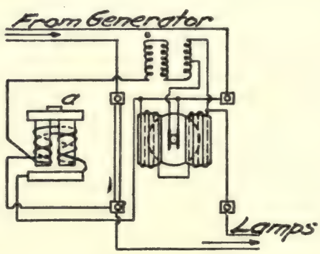


Fig. 402.

The drop in voltage caused by its presence in the circuit should not exceed 0.25 per cent.

It often becomes advisable to charge two rates, one for the short heavy peak loads, and another for the more steady average

loads. For measuring this kind of a load a two rate meter is used. This instrument will record the two periods of load separately, thus allowing the desired rates to be changed.

There are meters which record the ampere-hours instead of watt-hours, but these are not to be recommended for power house work.

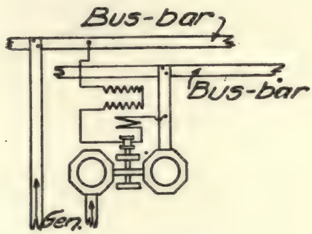


FIG. 403.

LIGHTNING ARRESTERS.

While some arresters work equally well on direct and alternating current circuits, the greater number do not. A very reliable arrester is the Garton, which is shown in Fig. 404.

The Westinghouse arrester is largely used on direct-current circuits up to 700 volts.

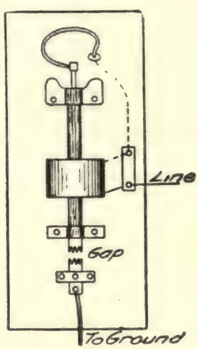


FIG. 404.

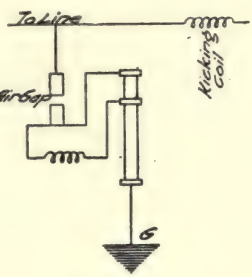


FIG. 405.

The General Electric Company puts out a magnetic blow-out arrester which has been largely used, especially on electric railway lines, etc., connections are shown in Fig. 405. These arresters are made for voltages up to 850. There should be one on each side of the line for added safety.

For alternating current practice the only satisfactory arresters are those using a series of cylinders, discs or spheres, having small air gaps between them and connected in series, the number depending on the voltage of the system.

The Wurts arrester, made by the Westinghouse, and the

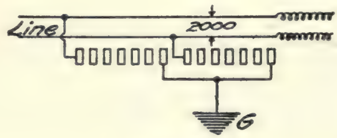


FIG. 406.

General Electric arresters are representative of this class. They are made for 1,000 volts, and when a line of higher voltage is to be protected enough are placed in series to sum up the required voltage, as in Fig. 406.

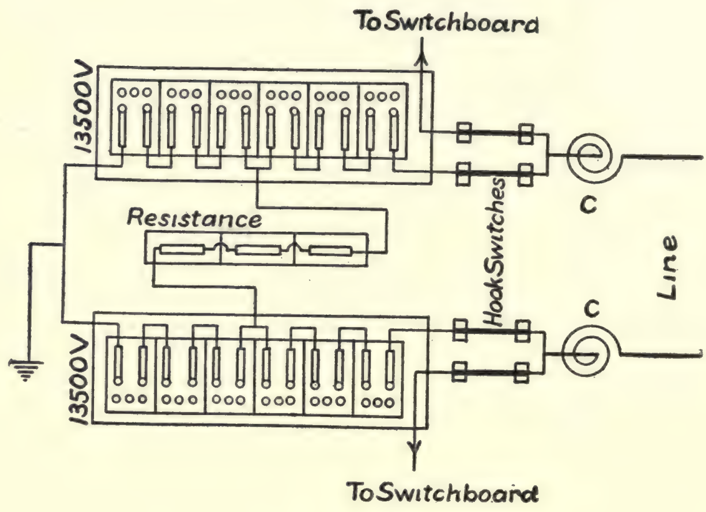


FIG. 407.

The General Electric arrester, shown in Fig 407, is installed for 10,000 volts. The choke coil for arresters may be made by coiling up about 150 feet of the line wire, making a coil about 15 inches in diameter. The ground indicated in the figures by must be very carefully made. A steel penstock or head

rack makes a good ground, or a galvanized iron pipe driven 12 or 14 feet into moist earth makes a good ground. All connections must be made as directly as possible and of ample size to carry the whole current.

In remodeling the Niagara plant Westinghouse low-equivalent arresters were installed, as shown in Fig. 408. These protect a 22,000 volt system. Each phase of each circuit has an arrester. They are mounted on marble boards and each board is mounted on castors, so that if damaged it may be wheeled out and replaced by a *spare*. The arresters are connected to the feeder line through fuses.

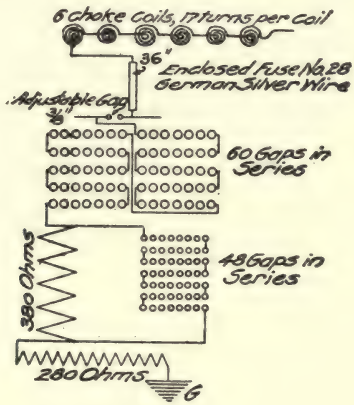


FIG. 408.

TRANSFORMERS.

The office of the transformer is to change the voltage of an alternating current circuit from one value to another, or to change the system from one phase to another. They come in all sizes, from the potential transformer used on switch boards for voltmeters to those of thousands of watts capacity.

In the large transformers above 25 kw. and below 75 kw. the case is usually filled with oil, which serves as an insulator. In some this oil is pumped in and out and cooled, while in others the case is given a large area to dissipate the heat by radiation. Still others are air cooled, a blower being constantly at work forcing air through the transformer.

In the power house we have to deal with the small transformers

used in connection with the switchboard instruments and the large step up transformers connected to the main lines.

The switch board potential transformers, etc., are only large enough to serve the instruments and are not intended to carry a load. They come all ready to place on the switch board and need no description.

The same type of transformer is used for any number of phases, a separate transformer being placed in each phase.

The following are the methods of cooling:

(1) Self-cooling dry transformer, made for voltages up to 15,000
(2) Self-cooling oil-filled " " " " " 80,000
(3) Cooled by forced air curents " " " " "
4) " " " water " " " " " "
(5) Cooled by both oil and water " " " " "

The over-heating of the transformer must be carefully guarded against and should never run above 80° degrees Centigrade. Thus, if the temperature of the room is 40 degrees Centigrade the interior of the transformer must not exceed it by more than 40 degrees Centigrade.

Types (1) and (2) are more expensive to build than the other types, as more iron has to be used, but are the best, all things considered.

The air cooled transformers are provided with air pipes connecting with a blower. In large plants they are set over air-tight rooms or pipes having openings in the ceiling through which the air passes to the transformers setting over the holes. The air pressure, amount of air, etc., is given in Table L.

TABLE L

DATA ON THE COOLING OF AIR-BLAST TRANSFORMERS.

Total capacity of trans-formers, kw.	Capacity of trans-formers, kw.	Ounce pressure per cu. in.	Cu. ft. air required per min. per transformer	Size of blower, inches.	Speed of blower, r.p.m.	Output of blower cu. ft. per min.	Power to drive blower, h.p.
300	50	.30	250	40	375	1,800	.25
900	100	.40	350	50	350	3,200	.60
1800	200	.50	600	C0	325	5,900	1.10
2700	300	.60	850	70	310	8,300	2.25
4500	500	.80	1,300	80	310	13,000	4.25
6700	750	.90	1,800	90	295	17,600	6.75
7500	1,250	1.00	3,000	100	280	23,600	12.00

The power consumed in cooling an air cooled transformer is about 0.3 per cent., or less. of the power delivered.

The approximate cost of transformers is from $4 to $7 per kw.

An oil cooled transformer may simply have its case filled with oil, or the oil may be circulated by means of a pump. The case is sometimes filled with oil and water caused to circulate around and through the case to cool the oil. About one gallon of water per minute per 300 kw. is necessary in this case. These artificially cooled transformers are smaller in size than those depending alone on the radiation of the cases, but have the disadvantage that should the pumps or blowers fail to act the temperature will run up. However, such transformers will stand such usage for an hour, and most faults can be remedied in that time. The pumping outfit should in all cases consist of two separate pumps.

Thin transformer cases should be avoided, as in case of fire they become punctured and the oil escapes. The casing should be at least $\frac{1}{4}$-inch thick.

TABLE LI.

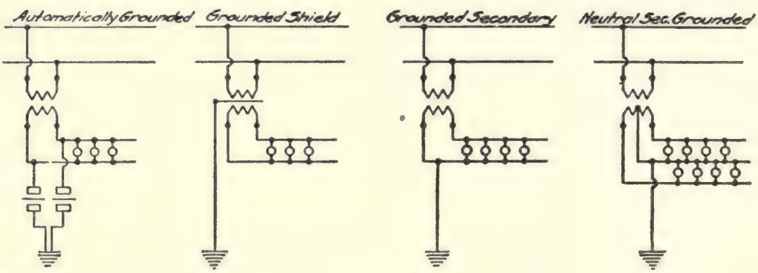
Watts capacity.	Core loss watts.	Full load copper loss in watts.	Regulation per cent.	Efficiency.				Weight in lbs.	Approx. cost.
				Full load.	$\frac{3}{4}$ load.	$\frac{1}{2}$ load.	$\frac{1}{4}$ load.		
600	25	16.7	2.93	93.5	92.9	91.1	85.2	70	
1,000	32	27.4	2.80	94.4	94.0	92.8	88.1	95	
1,500	38	37.5	2.63	95.2	95.0	94.0	90.3	125	
2,000	45	50.0	2.58	95.5	95.4	94.5	91.2	155	
2,500	50	54.0	2.23	96.0	95.9	95.1	92.1	195	
3,000	55	62.0	2.13	96.2	96.1	95.5	92.7	220	
4,000	63	85.0	2.19	96.4	96.4	95.9	93.6	270	
5,000	70	105.0	2.17	96.6	96.6	96.2	94.2	350	
7,500	110	147.0	2.50	96.7	96.6	96.2	94.6	470	
10,000	140	177.0	1.90	96.9	96.9	96.4	94.3	535	
15,000	175	272.6	1.90	97.2	97.1	96.8	95.1	850	
20,000	190	356.0	1.94	97.3	97.4	97.2	95.9	995	
25,000	220	460.0	1.98	97.3	97.5	97.3	96.1	1210	
30,000	250	495.0	1.81	97.5	97.7	97.5	96.3	1500	
40,000	390	590.0	1.65	97.6	97.6	97.4	95.8	1780	
50,000	460	690.0	1.48	97.7	97.7	97.5	96.1	1900	

The transformer is the *weak link*, and every precaution must be taken for its protection. If there is a break-down on the system the chances are three to one that the fault is with the

transformers. Lightning is the most dangerous foe and hence the latest and best arresters are none too good. A good plan, suggested by Professor Thomson, is that of interposing a metallic shield, connected to earth, between the primary and secondary windings.

Another safety device invented by Professor Thomson consists of a thin paper film between two metallic points, one of which is connected to the line and the other to the ground. This automatically grounds the line, though it is not at all certain to act.

One of the heroic measures for protection is to ground the secondary circuit. This is now permitted by the underwriters. All secondary circuits should be frequently tested for shorts. This is done by running test wires to the transformers and testing with a ground detector on the board. Each year a complete insulation test should be made of every transformer. Transformers are also inserted in the primary to add to the protection given by the arresters. The cases of all large station transformers should be thoroughly grounded to protect the workmen.



Figs. 409-412.

Figs. 409-412 show the most common methods of connecting the ordinary protective devices.

Transformers being almost invariably used for constant potential circuits they must be protected against heavy surges of current due to shorts on the line. The fuse, therefore, becomes a very important part of the transformer.

The Stanley Company, the General Electric Company, Westinghouse and many others make fuses especially for transformers. They have removable blocks, so that the fuses may safely be replaced. Transformers may be wound tor polyphase, but the

best practice is to use the single phase. Though requiring more iron the single phase arrangement permits the banking of the transformer, so that the disabling of one does not completely cripple the system.

In selecting the size and efficiency of transformers it must be borne in mind that it is not always best practice to install transformers of the highest efficiency. The more efficient transformers are higher priced, and when the difference between the costs of two transformers becomes greater than the power saved by the more efficient transformer could be sold for, the cheaper transformer would be the proper one to adopt.

The best power house practice is to use four, six, ten or more transformers banked together and having one transformer in reserve ready to be rolled into place. There is always danger of a transformer, especially the oil cooled type, catching fire, and it is quite important to provide for such contingencies. They should be in a separate room and have switches arranged on their cases so that by merely opening the switch the transformer is disconnected from the line and ground. It is a good plan to have each transformer mounted on a truck to facilitate quick movement.

Never, under any circumstances, install a cheap transformer. The standard transformers are quite reliable, but those which are not well known should be avoided. Table LII gives the proper sizes for transformers on three-phase motor circuits.

Transformers will carry as great an overload as the generators, so their normal rating should be the same as that of the generators.

Small transformers, below 25 kw., may be placed on poles. The common practice nowadays, however, is to use a few large transformers at sub-stations rather than numerous small ones mounted on poles at the houses. It is not good practice to mount transformers on the walls of houses, though at times this has to be done, in which case precautions must be taken to insulate them from the brick or woodwork.

Parallel connection is not advisable for small transformers where it can be avoided.

Transformers are often wound in sections, so that for high voltage they may be connected in series.

Where it is wished to increase the primary voltage on an old

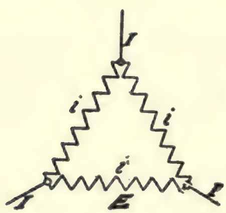
TABLE LII.

PROPER CAPACITY OF TRANSFORMERS FOR 3-PHASE MOTORS.

Capacity of Motor in h.p.	Capacity of transformers in kw.	
	Two transformers.	Three transformers.
1	0.6	0.6
2	1.5	1.0
3	2.0	1.5
5	3.0	2.0
7½	4.0	3.0
10	5.0	4.0
15	7.5	5.0
20	10.0	7.5
30	15.0	10.0
50	25.0	15.0
75	....	25.0

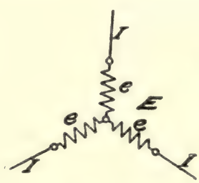
system, the primary windings of the old transformers may be connected in series.

In four-wire, two-phase systems, unless motors are to be operated, the transformers are connected to each phase in the same manner as for a single phase, but where motors are run there



$$i = \frac{I}{\sqrt{3}}$$

FIG. 413.



$$e = \frac{E}{\sqrt{3}}$$

FIG. 414.

must be one transformer for each phase, each having a capacity of one-half that of the motor or its rated overload.

In three-phase work two types of connections are used, namely, the delta, Δ, and the star, Y, connections. The relative voltages and currents for 1:1 ratio are given in Figs. 413 and 414.

Transformers may be used as phase changers (see Fig. 415). That is, a two-phase generator may be installed in the power house and the current changed to three-phase for transmission. This might be advisable where a part of the load is near by. This is the system at Niagara and in Fig. 415 are shown the connections for changing from two-phase to three-phase and then from three-phase back to two-phase.

One transformer should have a transformation ratio of 100:10 and the other 100:8.67. This latter ratio for small transformation is, in practice, made at 100:9. If the two transformers are wound the same or are interchangeable they must have a combined capacity of 12 per cent. more than the load. The 100:9 transformer is called a teaser and need be but 4 per cent. larger than would be normally required. For transforming the phases two transformers are usually used, which are made especially

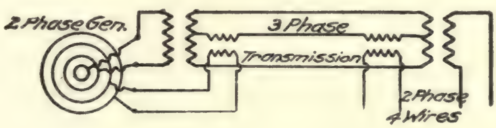


Fig. 415.

for the work, having two connections, one giving 50 per cent. and the other 86.7 per cent. the full voltage. In this case either transformer may serve as the teaser.

Storage Battery.

The storage battery is seldom used essentially in its storage sense, its cost prohibiting this, but invariably in the capacity of a regulator. Its importance is in almost direct proportion to the rapidity and magnitude of the fluctuations. The cost of a storage battery plant is high, but in all cases of engineering the only disideratum should be the relation between the total first cost and the future dividends, and when so considered the use of a battery plant will usually become a necessity for the following reasons:

(1) The capacity of the generating units required to carry the abnormal peak loads is decreased, thus minimizing the first cost of the plant.

(2) The maintenance of better voltage regulation on the whole system.

(3) The creation of a reserve force which may be utilized to carry a minimum load for a short time to permit of repairs, etc.

(4) A reduction in the annual cost of producing a unit of energy. A generating unit, when run at full load, is from 10 to 30 per cent. more efficient than when running on such loads as are common to all power plants not equipped with batteries. The wear and tear on a plant carrying a fluctuating load is quite severe, and plays an important part in the depreciation item. Especially is this true of a turbine plant where the action of a powerful governor on the gates causes great wear and vibrations.

(5) Increase of the capacity of transmission lines by carrying the peak loads at the point of use, thus making the line only carry the normal load.

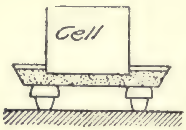


FIG. 416.

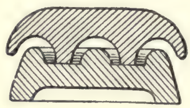


FIG. 417.

The most prominent makers of batteries are the Electric Storage Battery Company of Philadelphia and the Gould Storage Battery Company of New York. The Electric Storage Battery Company build the *chloride* battery and the Gould Company the *Gould* battery.

For central station use, the cells are best made in the form of lead lined wooden tanks, each cell or tank lining made somewhat longer than necessary to hold the plates first installed, so as to allow more plates to be added later on if found advisable. Being receptacles for the storage of energy under electrical pressure the cells themselves must be thoroughly insulated from the earth. This may be accomplished by placing each cell in a box of sand as in Fig. 416, and setting the box on four oil insulators, shown in Fig. 417, or, as is most frequently done, on porcelain insulators.

The room must be provided with special ventilation to carry off the acid fumes. The frame work supporting the cells must

be very strong, of wood and painted with an acid-proof paint. The plates contained in the cell constitute the *element*. The connections are all made by burning the terminals together.

Great care must be taken to get pure acid and distilled water. The capacity of storage batteries is given in ampere-hours or watt-hours under a certain rate of discharge. Every battery has its most efficient rate and if this is exceeded the capacity is lessened. If a cell can deliver 100 amperes for 10 hours on normal discharge its capacity will be 1000 ampere-hours, but if discharged in five hours its capacity may only be 800 ampere-hours. For this reason a sufficient number of batteries should be installed to carry at least 75 per cent. of the peak on a normal discharge.

The watt-hour capacity is more trustworthy than the ampere-hour, for while the latter efficiency may be as high as 95 per cent. the former is seldom above 70 per cent. to 80 per cent.

The new Edison battery weighs 57 pounds per kw.-hour capacity, and has a voltage per cell of from 1.1 to 1.5 volts. A battery having a capacity of 1000 kw.-hour capacity will, if all cells are on the same floor, take up about 100 square yards.

The largest cells may consist of from 80 to 100 plates. A single cell, such as used in the power house, gives from 6 to 7 ampere-hours.

Storage batteries must be charged at a slightly higher voltage than they are discharged at. This charging voltage averages from 2 to 2.5 volts per cell, and the discharging voltage will average from 2.5 to 1.8.

While charging, the voltage is gradually increased so that two cells may require 40 volts at the start and 50 at the finish. This is usually done by means of the booster or cutting-out resistance from the generator field. Without some regulating device the voltage delivered to the line would drop off from $20 \times 2.5 = 44$ volts to $1.8 \times 20 = 36.0$ volts.
To counteract this drop cells are added so that they may be thrown in one after another to keep up the pressure. These are called *end cells*.

Cells should never be completely discharged, 1.8 volts being the minimum voltage of the discharge.

To get the desired voltage for any system enough batteries are placed in series to give it: thus:

$$\frac{\text{Desired Battery Voltage}}{\text{Minimum voltage of cell.}} = \text{number of cells.}$$

Then to get the desired current groups of the cells in series as given by the above formula are connected in parallel, as in Fig. 418, where each cell has a capacity of 10 amperes.

Sometimes there are not enough cells to take up the voltage of the machine, in which case a resistance is placed in series with the cells. The amount of this resistance is found by dividing the difference between the voltage of the batteries and the generators, by the amperage of the battery.

The usual office of a storage battery makes it necessary to automatically cause the cells to be charged during hours of light load and discharged during peak loads.

In large plants, such as we are more especially treating, the regulation of the battery must be more rapid than hand regulation, and therefore a *booster* is used. This booster is a small

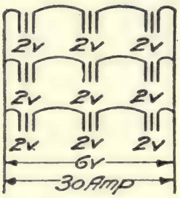


FIG. 418.

generator usually driven by a motor, and its action is as follows:

The battery and the armature of the booster are in series. The booster field has two windings, one is a fine wire shunt, and the other a few turns of heavy wire in series with the main line. The field current may be adjusted by means of a shunt rheostat. The effect of the booster is that of a number of cells added in series with the battery.

The shunt and field coils of the booster oppose each other in such a way that on normal discharge there is no e.m.f. generated. But when the line current falls below normal, the shunt coil, excited by the battery, takes effect and the booster delivers an e.m.f. which aids the generator in charging the batteries. When the line current is above normal due to a heavy load on the line, the series coil takes effect and forces the battery into helping carry part of the load.

The electrolyte, consisting of sulphuric acid and water, must be of the purest ingredients. The water is placed in the cell first and the acid poured slowly into the water. After the mixture has cooled and within two hours, the plates are placed in position, connected, and the batteries slowly charged. It is advocated by some makers to charge the batteries for the first time at about one-third the normal rate. Charging at a higher rate than recommended by the makers should never be attempted A little overcharging does no harm, but results in a waste of current.

When the cells are fully charged the following facts are apparent: Number of ampere-hours, *i.e.*, the product of the reading of the ammeter and the time comes to the desired amount; the voltage reaches the maximum; the positive plate becomes very dark; *gasing* takes place, that is, gas is given off, making the electrolyte boil. When this last phenomenon has gone on 10 or 15 minutes the battery is charged.

To prevent the escape of the gas the electrolyte is often covered an inch deep with paraffine and an inch-hole bored through to prevent the accumulation of pressure. Every cell must be easily accessible for examination and the plates and electrolyte frequently inspected. The plates become buckled in time and particles of the paste fall out and lodge between them. For this reason, frequent inspection is necessary to prevent short circuiting of the plates. Each cell must be thoroughly insulated from the earth. Cells should not be left standing for any great length of time without being charged, else sulphating will take place. Sulphating causes a scale to form over the plates, especially on the positive plates, reducing the capacity of the cell and causing a buckling of the plates. Sulphating is removed by carefully scraping the plates, after which they are charged at a slow rate for some time. If a storage battery is to be put out of use for any great length of time, they should be fully charged, the electrolyte drawn off and the cells then filled with pure water. They should then be discharged at their normal rate. After the water has stood in the cells for some 48 hours it is then drawn off and the battery will remain in good condition. If the plates become slightly buckled they may be straightened by pressing, not pounding, between two boards.

In Fig. 419 and 420 are given two load curves which were

taken at a large lighting and power station. The curve in Fig. 419 is a representative winter curve, and that shown in Fig. 420 was taken in the spring. The double hatched portion shows

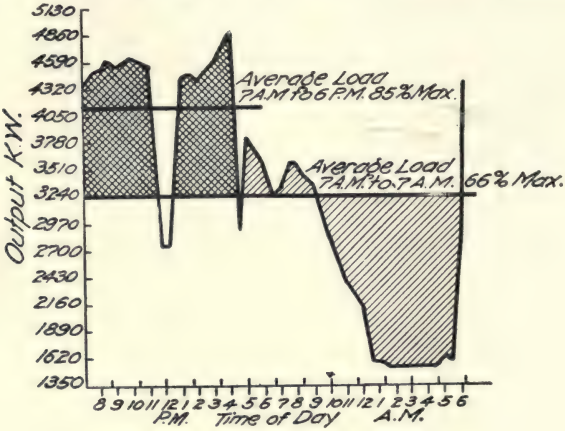


Fig. 419.

the load carried by the battery, and the single hatched shows the charging load of the power plant. The battery was of the chloride type, having a 10,000 kw.-hr. capacity.

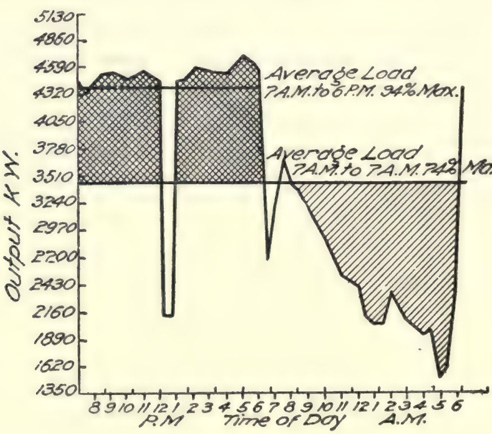


Fig. 420.

It will be seen that the maximum peak load amounts to 1600 kw. above the average. That is, had there been no battery used, 1600 kw. more power would necessarily have been generated by

machinery. Aside from the first cost, interest and depreciation of all this added machinery, it would have worked at full load only a small part of the day, so that its efficiency would have been very low. If the power had been generated by steam the boilers necessary for the peak loads would all have to be heated up and then allowed to cool down—a very costly and injurious thing to do. The use of the storage battery, therefore, equalizes the load and permits a lesser number of generating units to work at their most efficient output.

In Fig. 419 it will be seen that the batteries could have easily carried all the load there was between 1 and 5 a.m., if it had been necessary for a short time. However, if the battery had been installed with this point in view more battery power would have been required, as it is during those hours that the batteries must be charged.

The efficiency of a storage battery being about 70 per cent. the power of the machinery must be so proportioned that the double shaded portion of the power curves is 70 per cent. of the single shaded portion, *i.e.*, the battery should have a capacity 30 per cent. larger than the double shaded portion.

In designing a plant, curves should be drawn approximating as nearly as possible what the actual practice will be, and the battery and machine capacity worked out from them; then after the plant is in operation the curves may be brought to the desired form by regulating the charges for current.

To generate the 1600 kw. by steam would cost, for machinery, about $130,000, and the added yearly maintenance cost of the machinery would be about as follows:

Depreciation at 8 per cent. on $130,000 worth of machinery....................................	$10,400
Interest and insurance on $130,000 worth of machinery	8,000
Added cost of operating $130,000 worth of machinery..	4,000
2 per cent. added depreciation due to operating 4860 kw., or $390,000 worth of machinery on uneven loads	7,800
	$30,200

To get the watt-hour capacity of the battery multiply the hours the load is on by the average watts: thus take the case of the first peak in curve, Fig. 419; the average time is 4¼ hours

and the average load is about 1000 kw., which makes 4250 kw.-hrs., as the capacity required for that part of the load. Each peak is figured in the same way and the sum multiplied by 1.3 gives the necessary capacity of the battery.

In the above case the required 10,000 kw.-hr. battery would cost $40,000 and the yearly cost of operation, depreciation, interest, etc., would be about $4000. Several companies guarantee the maintenance to be 5 per cent. or less.

Fig. 421 shows the load curves on a large central station plant for a week day and for Sunday. Taking the week day card: The station was provided with a 1000 kw. unit which was thrown on to the load at 12 a.m. It carried the load with good efficiency until about 7 o'clock, when another, a 3500 kw., unit was thrown

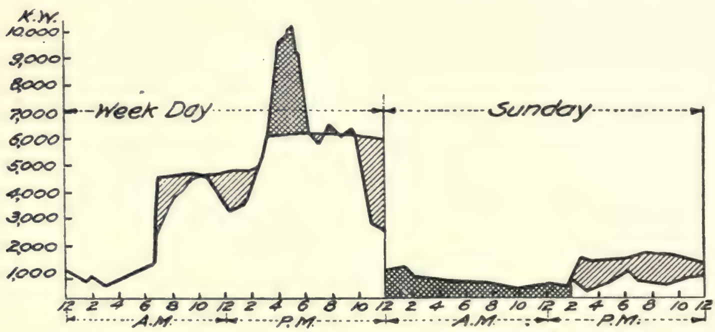


FIG. 421.

in. At this time the ammeter did not show up a full load so the storage battery was connected and charging commenced. The ammeter showed the load was increasing on the line, so at 2.30 a 1500 kw. unit was put into service and the battery thrown out until 3 p.m., when the battery was put into the line. The three units kept up their load until midnight by feeding the battery as shown. In this case the battery saved 4000 kw. capacity in machinery. The Sunday card shows how the battery carried the entire load from 12 midnight until 2 p.m.

In hydraulic transmissions the battery is placed at the far end of the line at the center of distribution. For alternating currents a motor generator is required to change to direct current, in which case an attendant is required constantly at the sub-station. But where the transmission is by direct current only an occa-

sional visit to inspect the battery is necessary, the booster being located at the power house.

The location of the battery at the consumer's end of the line adds another valuable feature to the battery installation. It permits the use of a smaller feeder, or for the same size of wire reduces the line drop. This is because the peak current is never sent over the line, the battery supplying all excesses. Old plants find this method a good one for increasing their capacity without changing the size of feeders or installing extra machinery.

On lighting loads a booster is used only to add a few volts to the generators, but on railway work the booster is so made that it regulates the batteries, causing them to charge and discharge.

An end-cell switch is used in connection with the boosters. This sometimes consists of a sliding contact caused to move along the threaded shaft by means of a small motor made especially for the purpose. A double throw single-pole switch is used to connect the battery either to the generator through the booster or directly to the line.

Motor Generators.

Motor generators are motor-driven generators. They commonly consist of a synchronous motor driving a direct current generator on either end of its shaft. The losses are those due to a synchronous motor and the losses of the two direct current generators.

The most common use for the motor-generator is for converting alternating current into continuous current for electric railways, though it is also frequently used for lighting and power. In this way power may be transmitted by three-phase current to great distances and then changed to direct current for the use of the consumer. It serves the same purpose as the synchronous converter, but differs from the converter in that it operates perfectly on the higher frequencies.

Frequency Changers.

The frequency may be changed to suit the requirements by using a frequency changer. This consists of a synchronous motor directly connected to an induction motor. The current to be changed is led into the stationary field winding of the induction motor, called the primary, and taken from the rotor called the

secondary. The frequency and voltage of the out-put will depend on the speed of the secondary. If the frequency is to be increased the induction motor must be driven backwards, and if the frequency is to be decreased it is driven forwards.

To change a frequency of 40 cycles to 60 cycles, the secondary would be run *backwards* at half speed, and to obtain 25 cycles from a 60 cycle current, the secondary would run *forwards* at about 0.4 times its rated speed. The capacity of the primary will have the same proportion to the out-put that the initial frequency has to the final.

In table LIII data for a frequency changer of 100 kw. capacity are given. The proportions would remain the same for other sizes.

TABLE LIII.

INDUCTION MOTOR AS FREQUENCY CHANGER.

Initial frequency.	Final frequency.	Primary capacity of ind. motor in kw.	Secondary capacity of syn. motor in kw.	Capacity of frequency changer in kw.	Speed of ind. motor r.p.m.	Speed of syn. motor r.p.m.	Direction and speed of running.
40	60	33	66	100	400	800	Half speed back.
30	60	50	50	100	400	800	Full speed back.
25	60	58	42	100	400	800	336 forward.
60	25	42	58	100	400	800	1920 r.p.m.back.
60	30	50	50	100	400	800	Half-speed ahead.
60	40	66	33	100	400	800	Half-speed ahead.

The efficiency would not, of course, be 100 per cent., as has been assumed, but would depend on the efficiency of the two motors used. In each there would be a loss of from 4 to 10 per cent., depending on the size and running conditions.

When driven backwards all mechanical losses are supplied by the driving motor, but when driven forwards the frequency converter may supply a part or all of the mechanical losses in the set.

The object of a frequency changer is to permit the use of synchronous converters on a system where a high frequency is demanded and to reduce line drop. A synchronous converter will not operate satisfactorily on the higher frequencies, about

25 being the best. Lighting service demands 60 cycles or more, and for long transmissions 25 cycles gives the smallest voltage drop. Therefore, to reconcile these oppositions recourse is had to the frequency changer.

Alignment of Machinery.

One of the most necessary instruments for this work is the architect's level. Such an instrument costs about $60. It should be a 14-inch level with a vernier for getting angles.

Reliable straight edges are indispensable. These should be three in number and 4, 6, and 8 feet long. Fig. 422 shows the best form.

Where the engineer has much shaft aligning to do it will pay to have the instrument shown in Fig. 423. The blades *G* and frame are made of tool steel. By turning the thumb wheel *C* the screw *H* is revolved in the nut at *F*. The screw turns in *D*, which moves up and down with it. *E* is another nut in which the

Fig. 422.

left-handed thread on the screw works. In operation the blades are placed astride the shaft and the screw run down till it just touches the top of the shaft to be aligned. As *H* moves downward the nut *E* moves upward. In this way the pivot of the wings *B* always remain the same distance from the center line of the shaft. This makes it very handy where the line shaft consists of several different sizes. In leveling the point *A* is brought into line with the tight wire. To get the shaft level in the horizontal plane the architects' level is set up and the pivot *B* is leveled at different points along the shaft.

Plumb lines should be very fine silk lines or steel wire. Plumb bobs should be heavy. Those filled with quick silver and weighing several pounds are the best.

After the shafting is all in place it should be given a very careful aligning. The loss of power in shafting is mostly due to poor alignment.

Where the bearings of a line shaft pass over masonry walls

the bearings are anchored to the wall, as in Fig. 424. The tapering boxes A should be well made, planed perfectly smooth on the outside and where placed in concrete well soaped or oiled. *After* the base of the bearing is placed and aligned the holes left by the removal of the boxes are filled with 1 to 1 cement-sand. At least $\frac{1}{2}$ inch is left between the bottom of the bearing base and

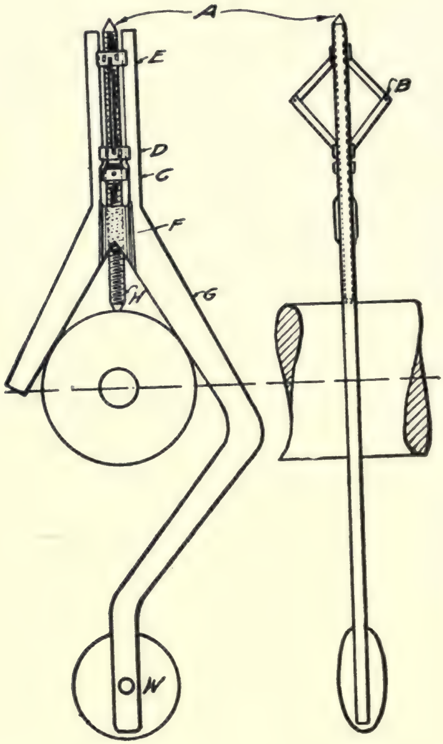


Fig. 423.

the top of the masonry. This is to allow for any small error in the first alignment and to permit pouring underneath the iron, and also into the bolt holes, a strong mixture of cement.

The tapering holes left by the boxes allow of shifting the bolt heads two inches or more each way before pouring the mixture. Fig. 424 shows one taper box removed, and the bearing in place ready to pour. The foundation bolts for engines and generators are made in the same way.

To avoid making a mistake in the spacing of the bolts a template should always be made, having the holes bored to exactly fit those in the base of the bearing or bed-plate. On this template the center lines may be marked to aid in aligning.

It is of extreme importance on extensive work to make accurate measurements. Instrument makers now make steel tapes which at a certain temperature and tension are exact. Tension handles are made by Keuffel & Esser Co., of New York, so that the engineer will know when the tension is exact. A scale is also made giving the correction for different temperatures. A 100-foot tape is about $\frac{1}{4}$ inch longer at summer heat than at the standard temperature of 62 degrees Fahrenheit. The engineer can have his tape certified at Washington by paying a dollar or so extra.

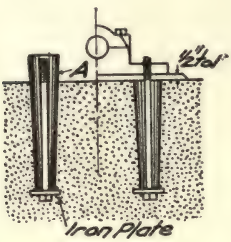


FIG. 424.

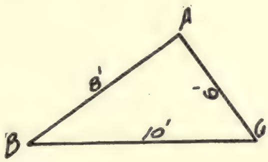


FIG. 425.

When working around power houses, the great foe to the steel tape is rust. The tapes may be nickel plated for from 50 cents to $2.

The common way to get a line at right angles to another is to measure off certain distances as $A\ B$ and $A\ C$ (Fig. 425), and then to make the distance

$$B\ C = \sqrt{A\ B^2 + A\ C^2}$$

The distances usually taken are 6 and 8, then

$$\sqrt{6^2 + 8^2} = 10$$

Where great accuracy is required greater distances may be taken. There should be a vernier on the level, in which case it may be used for getting right angles.

CHAPTER VIII.

POWER TRANSMISSION

There are in general four ways of driving machines from turbines, namely: direct connection; gears and shafting; belting; and rope drive.

Couplings.

The line shaft is divided up so that there are as many lengths as there are pinions. Heavy shafts are seldom longer than 20 feet. Couplings are used to connect the various pieces. These may be the plain disc coupling, the plate coupling or the compression.

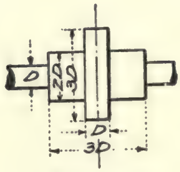


Fig. 426.

The proportions of a disc coupling are given in terms of the shaft's diameter, in Fig. 426. The size and number of bolts used to hold the halves together may be found from:

$$\frac{\text{horse power transmitted x 33,000}}{\text{Velocity of one bolt, ft. per min.} \times N \times 6000} = \text{area of one bolt.}$$

N = number of bolts.; 6000 = safe shearing strength per square inch of bolt.

Example: There is 500 h.p. to be transmitted. Shaft speed 225 r.p.m. Diameter of circle of bolt centers 16 inches. $N. = .6$

bolts. The velocity of the bolts is 884 feet per minute, therefore,

$$\frac{500 \times 33{,}000}{884 \times 6 \times 6000} = .52 \text{ square inches.}$$

The bolt of standard size nearest to this area is $\frac{7}{8}$-inch.

In connecting the generators a coupling is often used which has a little give-and-take motion called a *wabbler*, so that if the line shaft and generator are not quite in line there will be no binding. (Fig. 427.)

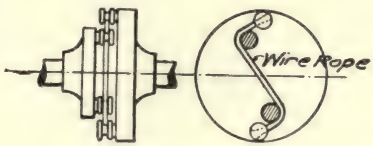


FIG. 427.

A jaw clutch coupling, Fig. 428, is a handy arrangement where it is necessary to uncouple frequently and quickly.

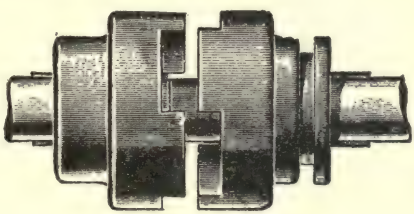


FIG. 428.

FRICTION CLUTCHES.

The friction clutch is a form of coupling which can be thrown in while one shaft is at full speed. They are made in all sizes up to several thousand horse power.

Clutches should be used only where absolutely necessary, as they are a weak link in the chain and get out of order easily. The bushings should be of bronze and self-oiling. The power transmitted by a clutch is proportional to the speed.

Keys.

Referring to Fig. 429, all dimensions being given in inches:

$$\text{for } D < 2,\ f = \frac{D}{12}+\frac{1}{16} \text{ and } b = \frac{D}{7}+\frac{1}{16}$$

$$\text{for } D > 2,\ f = \frac{D}{10}+\frac{1}{8} \text{ and } b = \frac{D}{5}+\frac{1}{16}$$

Where the pinions are slid on the shaft there should be two keys, one on each side of the shaft. They should fit snugly, but loose enough to permit the gear being slid back by hand. Screws are used to hold the key and gear in place.

Fig. 429.

Quill Shafts.

Fig. 430 shows an arrangement by means of which a gear or machine may be thrown out without affecting the line shaft. Thus two generators may be placed end to end on the same shaft, the generator next to the turbines being attached to the quill. Then this generator may be thrown out without stopping No. 2. No. 2 may be uncoupled, or un-clutched, without stopping No. 1. While certain conditions often make a quill advisable, it should be avoided where possible, as it introduces complications.

Shafting.

In the every day transmission of power by shafting a large per cent. of the power is lost due to poor design. If the shaft springs, not only is the friction of the bearings increased, but also that of the gearing. Shafting should always be calculated for bending moments and torsional moments.

The curves in Fig. 431 will quickly give the proper size of shaft for safe tensile strength of 7500 and shearing of 6000 pounds per square inch. Fig. 432 is for the purpose of getting the size for any other strength. Thus, if we find by Fig. 431 that a 6-inch shaft is required and we wish to know the proper size for

a safe strength of 20,000 pounds per square inch; following up the vertical ordinate from 6, Fig. 432, it strikes the 20,000 pound curve at the horizontal line which indicates a $4\frac{1}{2}$-inch shaft.

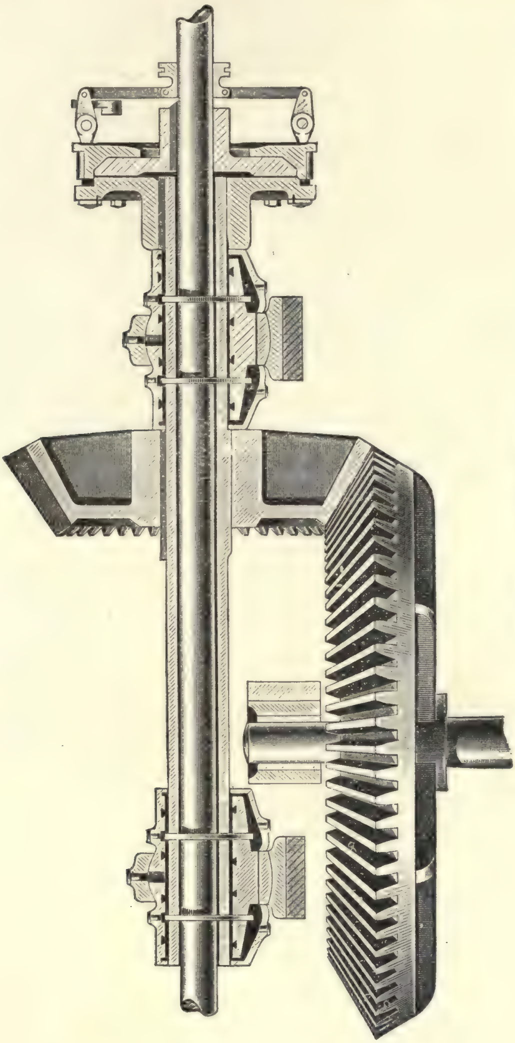


FIG. 430.—Quill Shaft.

One of the largest of manufacturing plants some time ago, while building their plant, guessed at the size of some shafts on the heavy conveyors. The designer had figured them for the

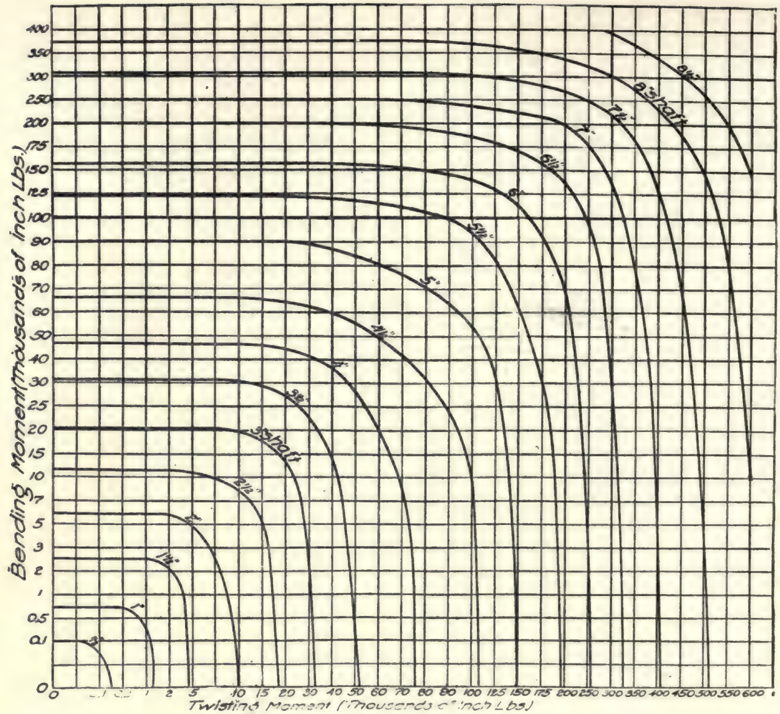


FIG. 431.—Curves giving proper size of shaft for given twisting and bending moments.

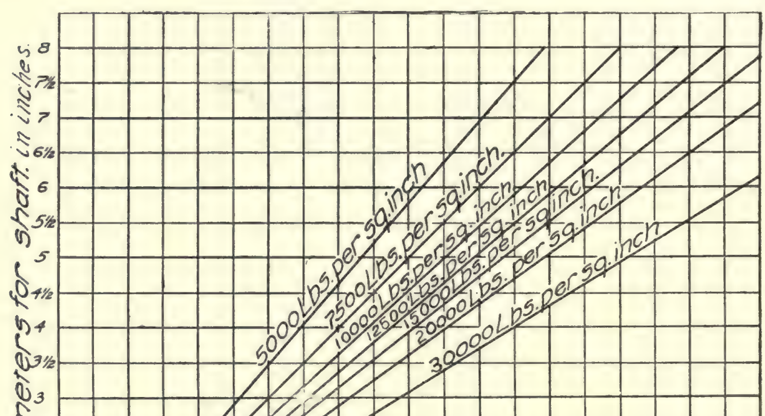


FIG. 432.—Diagram for getting new diameters when shaft has been figured from Fig. 431

torsional moments alone. When the plant started up six of the 24 shafts broke, and the author learned from good authority that the total cost caused by the accident amounted to over $20,000.

Fig. 433 gives the proper allowance for different fits. These curves may be used for obtaining the size of bore in the gears or pulleys.

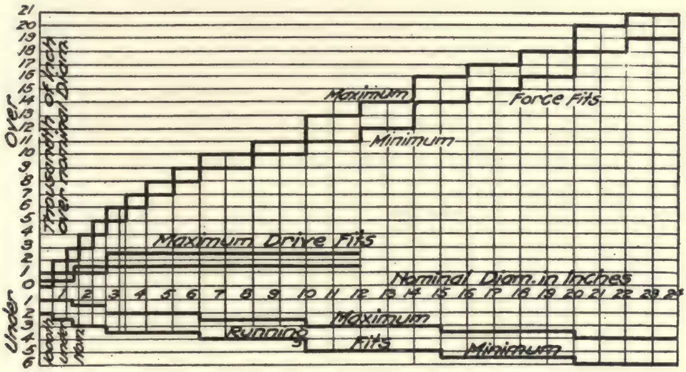


FIG. 433.—Diameters of shafts for various fits.

The following formulas, given by Thurston and modified by Jones & Laughlins, will be found fairly safe, though where first class work is desired they may sometimes give a shaft too small.

For head shafts well supported against springing

$$\text{H.P.} = \frac{d^3R}{125}; \quad d = \sqrt[3]{\frac{75\ \text{H.P.}}{R}} \text{ for iron.}$$

$$\text{H.P.} = \frac{d^3R}{75}; \quad d = \sqrt[3]{\frac{75\ \text{H.P.}}{R}} \text{ for cold-rolled iron.}$$

For line shafting hangers 8 feet apart

$$\text{H.P.} = \frac{d^3R}{90}; \quad d = \sqrt[3]{\frac{90\ \text{H.P.}}{R}} \text{ for iron.}$$

$${}^*\text{H.P.} = \frac{d^3R}{50}; \quad d = \sqrt[3]{\frac{50\ \text{H.P.}}{R}} \text{ for cold-rolled iron.}$$

For transmission only, no pulleys; or short counters

$$\text{H.P.} = \frac{d^3R}{50}; \quad d = \sqrt[3]{\frac{50\text{H.P.}}{R}} \text{ for turned iron.}$$

$$\text{H.P.} = \frac{d^3R}{30}; \quad d = \sqrt[3]{\frac{30\ \text{H.P.}}{R}} \text{ cold rolled iron.}$$

where d = diameter of shaft in inches and R = r.p.m.

*Is proper for turbine line shafts where $S = 10{,}000$ pounds per square inch.

Gears.

The most common method of driving, aside from that of direct connection is by means of gearing. By its use the speed of the machine may be made higher or lower than that of the turbine. Sometimes the ratio is as high as 4 to 1, though this is uncommon. The limiting factors are the peripheral speed of the cogs and the number of teeth on the pinion; the former must not exceed 1800 feet per minute for iron gears and 2400 per minute for an iron gear running on one with wooden teeth. These are limiting values and lower speeds should be used where possible.

The makers of gear wheels publish lists of their patterns, and in selecting the gears one must be governed by these; taking the nearest patterns.

There are three types of gears used for turbine connections, namely, mitre, bevel and spur.

Mitre and bevel gears are the same, the mitre being a bevel gear with the face at an angle of 45° with the line of shaft. That is, both gears are of the same size.

A spur wheel is one having the face of the teeth parallel with the shaft. One of a pair of heavy gears should always be a mortise gear, *i.e.*, a cast-iron frame with wooden teeth.

There are two kinds of teeth, cyclodial and involute.

The former type is most common for ordinary gear wheels, while the latter is mostly used for racks, etc.

The *circular pitch* equals the length of the *arc* in inches between the centers of two adjacent teeth.

The *diametral pitch* is given by the number of teeth per inch of the diameter of the pitch circle.

When starting the design of a gear decide on the pitch. The number of teeth in the wheel should not be divisable by the number in the pinion.

When the pinion (smallest gear) is driven by the wheel the number of teeth in pinion should not be less than eight. When the wheel is driven by pinion the number of teeth in the pinion should not be less than ten. Having selected the pitch and knowing the revolutions of the gear and velocity along the pitch circle which is considered good practice, the pitch diameter is found, and the teeth laid out.

Rule: To ascertain the revolutions of gearing multiply the

number of cogs in one by its number of revolutions and divide the product by the number of cogs in the other; the quotient will be the number of revolutions of the driven.

Rule: To ascertain the number of cogs in one, the number of its revolutions and the number of cogs and revolutions of the other being known, multiply the number of cogs in the latter by the number of its revolutions, and divide the product by the number of revolutions of the former; the quotient will be the number of cogs in the former.

Rule: Ascertain the pitch diameter of cog gearing, multiply the number of cogs by the number of thirty-seconds of an inch in the pitch and divide by π.

Example: A pitch of two inches has sixty-four thirty-seconds of an inch; say the wheel has 120 cogs; $\frac{120+64}{32\times\pi}$ gives 76.39 inches, the diameter of the pitch line.

To determine the horse power which any gear-wheel will transmit, four facts must be known, namely:

The kind of wheel, whether spur, bevel, spur mortise or bevel mortise; the pitch; the width of tooth called *face*; the velocity of pitch circle in feet per second.

Generally the fourth fact is not known. But it can be found if the pitch diameter of the wheel (in inches) and the number of revolutions per minute are given, for it can be obtained from them by the following rule:

Rule 1. Given the pitch diameter in inches and the number of revolutions per minute; to find, the velocity of pitch line in feet per second.

Multiply the *pitch diameter* (in inches) by the *number of revolutions per minute*, then divide the product thus found by 230; the quotient will be the velocity required.

Example: What is the velocity of the pitch circle of a gear-wheel in feet per second, the pitch diameter = 43 inches, revolutions per minute = 125.

$43\times125\div230 = 23.4$ feet per second.

For heavy gears subject to constant wear the pinion should have no less than 15 teeth. Gears may be designed having a large factor of safety and yet work at a pressure which will soon *wear* them out. In one of the most noted establishments in the United States the practice is to allow very low pressures

on the teeth. For instance, a cut iron tooth of 12-inch face and velocity of 1000 feet per minute has a pressure per inch of face of 125 pounds. A similar tooth with velocity of 600 has a pressure of 250 pounds. This, too, is for gears running entirely in oil.

$$\text{horse power of one tooth} = \frac{\text{press. at pitch circle} \times \text{vel. ft. per min.}}{33{,}000}$$

The pressure exerted at pitch line in pounds by one gear acting on another tending to produce rotation, is found thus:

$$\frac{33{,}000 \times \text{horse power}}{\text{Vel. of tooth at pitch circle in ft. per min.}}$$

Thus a gear 60 inches in diameter, transmitting 100 horse power to the pinion, exerts at the pitch circle the pressure

$$= \frac{33{,}000 \times 100}{15.70 \times 150} = 1400 \text{ pounds,}$$

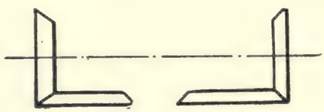


FIG. 434.

15.7 being the circumference of a 5-foot circle. All this pressure is considered as acting on a single tooth. In the above, if the tooth is 10 inches wide the pressure per inch of tooth equals 140 pounds. This would be a very moderate pressure. In heavy gears used continually and where more or less grit gets into them, as low a pressure as 100 pounds per inch of tooth is used. Such gears should be incased in dust proof cases and run in oil.

Common practice gives $y = 240\,p\,b$ where y is the pressure on the tooth as found above; p the pitch and b the breadth of the tooth = ten inches in above example. $y = 200\,p\,b$ is much better practice for wear. p = circular pitch.

Where two or more gears are placed on the same shaft they should be so placed that they thrust in opposite directions, thus neutralizing the effect. (See Fig. 434.)

For *strength* of a gear, $W = s\,p\,f\,y$, where s = values in Table LIV, p = circular pitch, f = width of tooth, y = the values given in the Table LV.

The width of the tooth is generally 2 to 3 times the pitch p, but may be greatly increased to reduce the pressure per inch of tooth. Thickness of the rim of gear below root of tooth equals depth of tooth.

TABLE LIV (Lewis).

SAFE STRESS ON TEETH PER SQUARE INCH OF MATERIAL.

Velocity of teeth in ft. per min.......	100	200	300	600	900	1200	1800	2400
Safe stress, s, cast iron.............	8,000	6,000	5,000	4,000	3,000	2,400	2,000	1,700
Safe stress, s, steel..	20,000	15,000	12,000	10,000	7,500	6,000	5,000	4,300
†Safe stress, s, wood	5,000	4,000	3,000	2,500	2,000	1,500	1,300	1,000

† Tredgold.

TABLE LV.

VALUES OF y.

No. of teeth.	Factor of Strength y.			No. of teeth.	Factor of Strength y.		
	Involute 20° obliquity.	Involute 15°, and cycloidal.	Radial flanks. 15°		Involute 20° obliquity.	Involute 15°, and cycloidal.	Radial flanks.
12	.078	.067	.052	27	.111	.100	.064
13	.083	.070	.053	30	.114	.102	.065
14	.088	.072	.054	34	.118	.104	.066
15	.092	.075	.055	38	.122	.107	.067
16	.094	.077	.056	43	.126	.110	.068
17	.096	.080	.057	50	.130	.112	.069
18	.098	.083	.058	60	.134	.114	.070
19	.100	.087	.059	75	.138	.116	.071
20	.102	.090	.060	100	.142	.118	.072
21	.104	.092	.061	150	.146	.120	.073
23	.106	.094	.062	300	.150	.122	.074
25	.108	.097	.063	Rack	.154	.124	.075

It is often advisable to calculate gears by diametral pitch. Thus, if the centers of two shafts, upon which it is desired to place different sets of gears having varying ratios, are fixed, use diametral pitch. Since diametral pitch is so many teeth per inch of diameter, by selecting a distance between centers which,

when multiplied by that diametral pitch, will give a whole number, any desired combination of gears can be used.

Suppose the distance between centers is 26.41 inches and the diametral pitch $p' = 3$, we have $N = p' \times 26.41 \times 2$ and $N = 156.86$ which is not a whole number. If the distance = 26.33, we have $N = 158$ as the number of teeth in both gears. If the ratio of the gears = 6 to 1 we have $\frac{168}{6} = 28$ teeth in the pinion and $168 - 28 = 140$ teeth in the gear. Any ratio may be selected, the only limitation being that the sum of all the teeth must equal 168; 26.66 inches would have worked out also. If the pitch was 4, then 26.25, 26.5, and 26.75 would have given a whole number, etc.

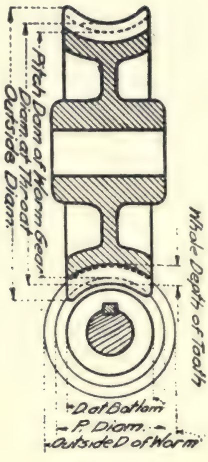


FIG. 135.

WORM AND GEAR.

The worm and gear is a construction which is used where a large transmission ratio is desired.

The best results are obtained when the worm is of hardened steel with polished double or triple threads, and having a ball thrust bearing, the gear being of bronze with *hobbed* teeth, the whole running in a bath of oil. Involute teeth are best. Use *circular* pitches in all calculations. The lead of a worm is its advance per revolution. The lead of a double thread worm equals twice its pitch.

To get the strength consider thrust of worm as acting on one tooth of a length equal the face. Solve as for other gears. The proper proportions are as follows:

Outside diameter of worm, single thread	4xpitch
Outside diameter of worm, double thread..........	5xpitch
Outside diameter of worm, triple thread..........	6xpitch

Face of worm gear wheel = 0.75 outside diam. of worm.

Pitch diam. of gear wheel = (no teeth x pitch) $\div \pi$.

Throat diam. of gear wheel = pitch diam. $+ 2 \div$ diametral pitch

Outside diam. of gear wheel = pitch diam. $+ 4 \div$ diametral pitch

Diametral pitch = $\pi \div$ circular pitch

Included angle of worm tooth = 29°

Whole depth of tooth of worm or gear = $.687 \times p$

HARNESS.

That part of the iron work supporting the line shaft of the turbine drive, is called the harness. The harness is made up

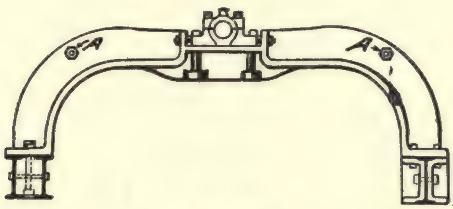


FIG. 436.

of the bearings, bridge-trees and the steel beams supporting the bridge-trees. Commonly the bridge-trees are made of cast iron, though they may be of wood or steel.

Fig. 436 shows a complete bridge-tree of a common type. Sometimes two *I* beams are used, as shown to the left, and at others one.

Fig. 437 gives a view of a pair of gears supported by timber bridge-trees. It will be noted that the bearing *A* is of necessity very short, $1\frac{1}{2}$ *D*, but the bearing *B* gives the necessary rigidity. The bearing *B* should be at all times out of water. This view also shows how the shaft is enlarged at the pinion. In this case the different turbines were thrown out of use by simply slipping the pinion out of gear. In Fig 436 two rods *A* are

shown. These run from bridge-tree to bridge-tree and tend to steady the entire harness. All bearing should be of the ball-and-socket ring oiling type. Roughly, a bridge-tree will cost $100, or about 4 cents per pound (without bearings).

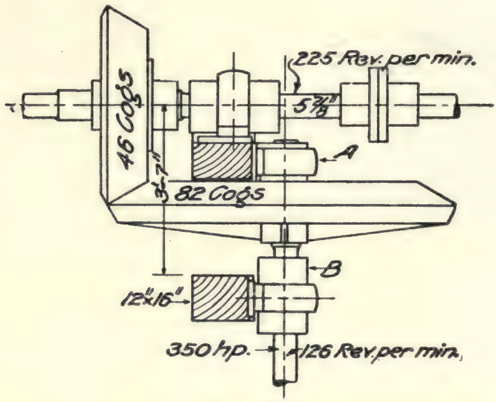


FIG. 437.

BELTING.

Modern practice is to dispense with belts wherever possible, substituting an electric, a compressed air or a rope drive. However, as they are still used to some extent, a brief treatment of the subject will be necessary.

For steady, hard usage in dry places a good leather belt is preferable to all others. Leather belts will stand rubbing, such as caused by crossed belts, shifters, etc. For damp places the rubber or gandy belt is used. The power transmitted by the rubber and leather belt is about the same for the same tension.

Ordinary belts will safely stand a tension of 45 pounds per inch of width for single belt and 75 pounds per lineal inch for double. This tension is exerted at the periphery of the pulley and becomes a measure of the power transmitted; thus a single belt 10 inches wide runs over a pully at a speed of 1000 feet per minute, therefore

$$\frac{1000 \text{x} 10 \text{x} 45}{33{,}000} = 13.7 \text{ h.p.}$$

The longer the belt the better, because a less tension has to be maintained, as the sag increases the arc of contact.

It must be remembered that the effective tension is the difference between the tension on the tight and loose sides when running with a load. Therefore, calculate the tension necessary to pull the load and make the tension on the belt when idle equal to the safe tension less this effective tension.

In selecting a belt it must be borne in mind that while the power transmitted is directly proportional to the tension, it is often a bad policy to get the necessary tension by merely tightening the belt or taking up the slack. On short spans it is better to get the necessary tension by adding to the weight of the belt, Fig. 438. This increases the sag and arc of contact.

For pulleys over 12 inches diameter use a double or triple leather belt or a correspondingly heavy rubber or cotton belt. The latter when 6 to 7-ply has an effective pull—6/7 that of first-class single ply leather.

FIG. 438.

Wave motion on the slack side and running from side to side of pulley under light loads is caused by too thin a belt. This wave motion wears out bearings, shafting and belts. Avoid *vertical* belts. The angle should be at least 45 degrees. Avoid such long heavy belts that the allowable tension is exceeded. While the tensions given in the tables are considered good practice by some of the most reliable manufacturers, Taylor claims that if half the tension is used the life will be increased about 2.6 times. The efficiency will also be increased and the life of the bearings.

For leather belts *always* place the hair side of the belt next the pulley as so placed it will transmit 30 per cent. more power than if the hair side is placed outside. For narrow belts run over small pulleys the distance between center should be at least 15 feet. For larger belts, say 6 to 12 inches, 20 to 25 feet and for the largest sizes 25 to 30 feet.

Whang leather lacing makes the best fastening especially

for small pulleys, but large belts should be made continuous by splicing and cementing.

Belts run best at high speeds (not more than 5000 feet per minute for single nor more than 4000 feet per minute for double leather belts). All belting should be laid out so that the *slack* side of the belt is on *top*, the pull being on the *lower* belt.

The idler should be placed as close as possible to the smallest pulley regardless whether it is the driver or the driven. An idler (or tightener) is absolutely necessary on vertical belts; speed should not exceed 5000 feet per minute.

A splendid splice for rubber belts is that shown in Fig. 439. The surfaces of one end of belt as a, a are given a coat of rubber dissolved in gasoline. The surfaces b, b of the other end are coated with rubber dissolved in bisulphide of carbon. Place in a press till dry.

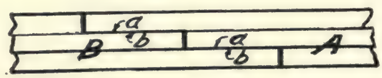


FIG. 439.

The horse power per inch width is found from the following formula:

$$S\,k = \text{horse power},$$

wherein S is the speed in feet per minute and $k = 0.001665$ for single leather belts and $k = 0.002666$ for double leather belts.

For special cases we find the proper width from the above formula and then multiply by the coefficient C, given in the following:

Double horizontal crossed belts....................... $C = 1.2$
Single vertical open belts.......................... $C = 1.8$
Double vertical driver............................. $C = 2.0$
Single horizontal, large driver to small pulleys......... $C = 1.2$
Double horizontal, large driver to small pulleys........ $C = 1.3$
Quarter turn single belts.......................... $C = 1.5$
Quarter turn double belts.......................... $C = 1.8$

Large belts transmit power with a loss of from 6 to 12 per cent. This includes the loss in the four bearings supporting the two pulleys.

Rope Transmission.

Rope transmission deserves a prominent place among transmitting devices. In cleanliness, efficiency and cheapness it is much superior to shafting and belts. It is adapted to the lightest or the heaviest power transmission.

MANILLA HEMP AND COTTON ROPES.

The usual transmission rope for long distances is of steel and consists of six strands having 19 wires to the strand. A hemp core is placed at the center to give pliability, but for short drives such as could be made with a belt a number of hemp or manilla ropes of three strands are used.

TABLE LVI (C. W. Hunt).

Horse Power of Manilla Rope at Various Speeds.

Diam. of Rope.	Speed of Rope in Feet per Minute.											Smallest Diam. of Pulley.
	1500	2000	2500	3000	3500	4000	4500	5000	6000	7000	8000	
$\frac{1}{2}$	1.45	1.9	2.3	2.7	3.	3.2	3.4	3.1	2.2	2.2	0	20 in.
$\frac{5}{8}$	2.3	3.2	3.6	4.2	4.6	5.0	5.3	4.9	3.4	3.4	0	24 "
$\frac{3}{4}$	3.3	4.3	5.2	5.8	6.7	7.2	7.7	7.1	4.9	4.9	0	30 "
$\frac{7}{8}$	4.5	5.9	7.0	8.2	9.1	9.8	10.8	9.3	6.9	6.9	0	36 "
1	5.8	7.7	9.2	10.7	11.9	12.8	13.7	12.5	8.8	8.8	0	42 "
$1\frac{1}{4}$	9.2	12.1	14.3	16.8	18.6	20.0	21.4	19.5	13.8	13.8	0	54 "
$1\frac{1}{2}$	13.1	17.4	20.7	23.1	26.8	28.8	30.8	28.2	19.8	19.8	0	60 "
$1\frac{3}{4}$	18.	23.7	28.2	32.8	36.4	39.2	41.8	37.4	27.6	27.6	0	72 "
2	23.2	30.8	36.8	42.8	47.6	51.2	54.8	50.0	35.2	35.2	0	84 "

TABLE LVII (C. W. Hunt).

Proper Sag of Manilla Rope.

Distance between pulleys, feet.	Driving side.	Slack Side of Rope.		
	All speeds.	80 ft. per sec.	60 ft. per sec.	40 ft. per sec.
40	4 inches	7 inches	9 inches	11 inches
60	10 "	17 "	20 "	23 "
80	17 "	28 "	34 "	39 "
100	24 "	44 "	53 "	62 "
120	35 "	63 "	75 "	88 "
140	46 "	86 "	105 "	117 "
160	60 "	111 "	135 "	168 "

TABLE LVIII.

PROPER TENSION ON SLACK PART OF ROPE

Speed of Rope, ft. per sec.	Diameter of Rope and pounds tension on slack rope.								
	½″	⅝″	¾″	⅞″	1″	1¼″	1½″	1¾″	2″
20	10	27	40	54	71	110	162	216	283
30	14	29	42	56	74	115	170	226	296
40	15	31	45	60	79	123	181	240	315
50	16	33	49	65	85	132	195	259	339
60	18	36	53	71	93	145	214	285	373
70	19	39	59	78	101	158	236	310	406
80	21	43	64	85	111	173	255	340	445
90	24	48	70	93	122	190	279	372	448

One or more grooves may be used the bottom of the groove being lined with wood or leather filling (Fig. 440), which lessens the wear on the rope. The wood filling is apt to get loose when

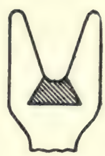


FIG. 440.

exposed to the weather, and even the leather if left idle will loosen up.

A filling consisting of alternate pieces of leather and rubber is the best.

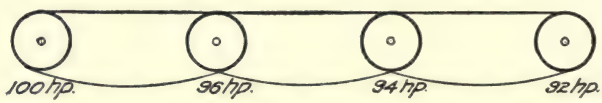


FIG. 441.

The efficiency of a rope drive is quite high, being about 96 per cent. in a single span drive and decreasing 2 per cent. for each relay or sub-division of the system as shown in Fig. 441.

When several grooves are used they may be turned out of

the solid metal as shown in Fig. 442, no filling being used and the ropes not touching the bottom. The iron must be smooth and free from all flaws.

TABLE LIX.

HEMP ROPE, THREE STRANDS.

Diameter of pulley, feet.	Size of Rope.		Strength		Weight per ft., pounds.	Length per lb., feet.
	Diameter, inches.	Circum., inches.	Breaking strength, lbs.	Safe strength.		
21	6	17.1	324,000	10,800	9.4	.1064
19	5½	15.7	272,000	9,070	7.9	.1266
16.5	5	14.25	225,000	7,800	6.52	.1533
14	4½	12.1	182,000	6,100	5.28	.1894
12	4	11.4	144,000	4,850	4.18	.2392
11	3¾	10.7	126,000	4,100	3.67	.2725
10	3½	10.	110,000	3,500	3.2	.3125
9	3¼	9.27	95,000	2,970	2.76	.3613
8	3	8.57	81,000	2,530	2.35	.4255
7	2¾	7.85	68,000	2,100	1.97	.5076
6	2½	7.14	56,200	1,800	1.63	.6135
5.25	2¼	6.43	45,500	1,420	1.32	.7575
4.25	2	5.70	36,000	1,100	1.04	.9615
3.4	1¾	5.10	27,500	900	.80	1.25
2.75	1½	4.28	20,200	630	.588	1.700
2.1	1¼	3.97	14,000	430	.407	2.457
1.5	1	2.86	9,000	280	.261	3.831
1.22	⅞	2.5	6,900	210	.200	5.000
.97	¾	2.14	5,050	150	.147	6.803
.74	⅝	1.78	3,500	100	.102	9.803
.53	½	1.43	2,240	75	.065	15.38
.34	⅜	1.07	1,260	......	.036	27.77
.18	¼	.71	560	......	.016	62.5

The horse power transmitted by one rope is

$$H = \frac{v\,D^2}{825}\left(200 \quad \frac{v^2}{107.2}\right)$$

where v is the velocity in feet per second, and D the diameter of rope in inches.

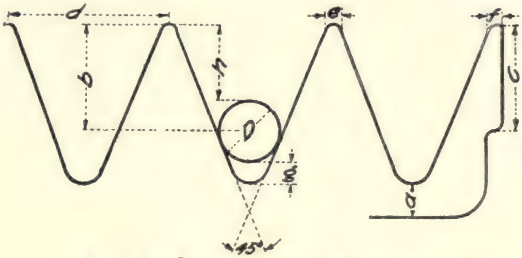
The tension on the slack rope when working is

$$t = \frac{(T-F)}{3} + F.$$

where T is the tension due to the transmitting of the power + the tension due to the weight of rope; F the centrifugal force,

$$F = \frac{p\,v^2}{32.2},$$

where p is the weight of one foot of rope in pounds and v the velocity of rope in feet per second. The tension T, due to the power is $\frac{33000 \times H}{V}$, when V is the velocity of rope in feet per minute. In solving the above equation t may be taken from Table LVIII in finding T.



$a = \frac{1}{2}D$; $b = \frac{3}{4}D + \frac{1}{16}$, $c = D$, $d = 1.6D$,
$e = \frac{1}{4}D + \frac{1}{16}$; $f = \frac{1}{4}D + \frac{3}{16}$; $g = \frac{1}{3}D$; $h = \frac{1}{4}D + \frac{1}{16}$;

FIG. 442.

Table LVII gives the sag on the slack side of the rope to produce the proper tension, or, the sag which would be produced by the tension given in Table LVIII.

The most efficient drives are made with a large number of small ropes. A 5/16-inch rope running over 30-inch sheaves is the best for drives up to 20 h.p., ½-inch for drives of from 20 to 40 h.p. and ¾-inch for 40 to 80.

Where the sheaves are of different diameters the ropes do not pull alike and therefore the angle of the grooves on the smaller pulley must be made less so as to get the same friction on both pulleys. On long out-door transmissions the large sheaves receive heavy side pressures from the force of the wind and must be made strong laterally.

It is advisable to place idlers to support the rope where on account of the space, the tension becomes excessive.

STEEL ROPES.

Steel ropes are largely used for out-door work. The sheaves or pulleys are the same as for the soft ropes though they are more often without filling.

As in the case of the soft ropes there must be a sufficient amount of friction between the sheaves and the rope to prevent slipping. This, having selected the proper diameter of pulley, is obtained by regulating the tension on the cable by means of a tail sheave and counter weight.

The coefficient of friction, f, for sheaves is given as follows:

	Dry	*Wet*	*Greasy*
Rope on a grooved iron pulley..........	.120	.085	.070
Rope on a wood-filled pulley............	.235	.170	.140
Rope on a rubber and leather filling......	.495	.400	.205

Then to find the proper weight W on the counter balance, $W = C\,P$, where $P = \frac{33000 \times H}{V}$, is the useful pull on the rope, and C is a constant depending on the above values of f and the number of ropes, N, used. C is given in Table LX.

TABLE LX.

VALUES OF C IN FINDING PROPER COUNTER WEIGHT.

f.	N. Number of ropes used on half laps.					
	1	2	3	4	5	6
.07	9.130	4.623	3.141	2.418	1.999	1.729
.085	7.536	3.833	2.629	2.047	1.714	1.505
120	5.345	2.777	1.953	1.570	1.358	1.232
.140	4.623	2.418	1.729	1.416	1.249	1.154
170	3.833	2.047	1.505	1.268	1.149	1.085
.205	3.212	1.762	1.338	1.165	1.083	1.043
235	2.831	1.592	1.245	1.110	1.051	1.024
.400	1.795	1.176	1.047	1.013	1.004	1.001
.495	1.538	1.095	1.019	1.004	1.001	.....

Example: There are 400 h.p. to be transmitted; 2-inch rope; speed of rope = 3600 feet per minute;

$$P = \frac{33000\text{x}400}{3600} = 3666 \text{ pounds useful pull on the rope.}$$

Find N, the number of ropes. The actual horse power transmitted may be found approximately from

$$H = 3.1\,D^2 \times v,$$

where D = diameter of rope and v = velocity in feet per second, therefore $H = 3.1 \text{x} 4 \text{x} 60 = 74.4$. But we have 400 h.p. to transmit, therefore,

$$\frac{400}{74.4} = 5.37 = n \text{ ropes.}$$

Taking 6 as the proper number and referring to Table LX for a wet rope on an iron pulley, $C = 1.505$, $W = C\,P$.

$$W = 1.505 \text{x} 3666 = 5517 \text{ pounds.}$$

TABLE LXI.

PROPER DEFLECTION FOR WIRE ROPE.

Span in feet	50	100	150	200	250	300	350	400	450
Deflection in inches	1.75	7	15.5	27.62	42.25	62.25	84.62	110.62	140

A less deflection than 3 inches corresponding to a span of 54 feet does not give satisfactory results.

The maximum tension on the rope occurs at the ends and is given by

$$T' = \frac{p\ l^2}{8h} + p\,h$$

when p is the weight of one foot of rope; l the horizontal distance between pulleys in feet, and h the deflection given in Table LXI. To this tension must be added that due to transmitting the power or the useful pull, as above found. The horizontal pull on the pulley is the useful pull + 2 T'. If the tension T' is the W in the above formulas there will be no counter weight required, and in all cases the counter weight should equal W minus T.

The sheaves should be of large diameter, as not only does the efficiency of the drive depend largely upon it, but also the wear of the ropes.

For ropes of 7-wire strands the pulley should have a diameter equal to 150 D; for 12 strands, 115 D; for 19 strands, 90 D, D being the diameter of the rope.

TABLE LXII.

HORSE POWER OF WIRE ROPES.

Diam. of wheel, feet.	No. of revs. min. of pulley.	Diam. of rope.	H.P.	Diam. of wheel, feet.	No. of revs. per min.	Diam. of rope.	H.P.
3	80	$\frac{3}{8}$	3	7	140	$\frac{9}{16}$	35
3	100	$\frac{3}{8}$	$3\frac{1}{2}$	8	80	$\frac{5}{8}$	26
3	120	$\frac{3}{8}$	4	8	100	$\frac{5}{8}$	32
3	140	$\frac{3}{8}$	$4\frac{1}{2}$	8	120	$\frac{5}{8}$	39
4	80	$\frac{3}{8}$	4	8	140	$\frac{5}{8}$	45
4	100	$\frac{3}{8}$	5	9	80	$\frac{5}{8}$	47
4	120	$\frac{3}{8}$	6	9	100	$\frac{5}{8}$	59
4	140	$\frac{3}{8}$	7	9	120	$\frac{5}{8}$	70
5	80	$\frac{7}{16}$	9	9	140	$\frac{5}{8}$	83
5	100	$\frac{7}{16}$	11	10	80	$\frac{11}{16}$	66
5	120	$\frac{7}{16}$	13	10	100	$\frac{11}{16}$	83
5	140	$\frac{7}{16}$	15	10	120	$\frac{11}{16}$	97
6	80	$\frac{1}{2}$	14	10	140	$\frac{11}{16}$	116
6	100	$\frac{1}{2}$	17	12	80	$\frac{3}{4}$	96
6	120	$\frac{1}{2}$	20	12	100	$\frac{3}{4}$	120
6	140	$\frac{1}{2}$	23	12	120	$\frac{3}{4}$	145
7	80	$\frac{9}{16}$	20	12	140	$\frac{7}{8}$	173
7	100	$\frac{9}{16}$	25	14	80	$1\frac{1}{8}$	145
7	120	$\frac{9}{16}$	30	14	100	$1\frac{1}{8}$	180

In figuring the strain on the rope the actual area of all the wires in the rope must be taken. Safe stress for iron rope is 24,640 pounds per square inch.

Steel ropes are preferable to iron. For a long transmission such as several thousand feet, select a larger rope than the tables call for and run it at a low velocity over large sheaves. Support the ropes on large idlers every hundred feet or so. Such a transmission is well suited to hydraulic powers where the factory is some distance away from the power house.

CARE OF THE ROPES.

All steel or iron rope must be frequently oiled with linseed oil, tar or any oil free from acid. It should be very carefully uncoiled from the shipping coil to avoid kinks.

The nominal diameter of the rope is the diameter of the circle which just encloses it, and this diameter must always be given in ordering.

TABLE LXIII.

Hoisting Ropes = 6 Strands of 19 Wires Each.

			Iron.		Cast-Steel.		Extra Strong Cast Steel.		Plow-Steel.		
Diameter in inches.	Approximate circumference in inches.	Estimated weight per foot in lbs.	Approximate breaking stress in lbs.	Maximum safe stress in lbs. = ⅓ ult. stress.	Approximate breaking stress in lbs.	Maximum safe stress in lbs. = ⅓ ult. stress.	Approximate breaking stress in lbs.	Maximum safe stress in lbs. = ⅓ ult. stress.	Approximate breaking stress in lbs.	Maximum safe stress in lbs. = ⅓ ult. stress.	Proper working load.
2¼	7⅛	8.00	156,000	52,000	312,000	104,000	364,000	121,333	416,000	138,667	Maximum safe stress less bending stress determined by Table LXIV.
2	6¼	6.30	124,000	41,333	248,000	82,667	288,000	96,000	330,000	110,000	
1¾	5½	4.85	96,000	32,000	192,000	64,000	224,000	74,667	256,000	85,333	
1⅝	5	4.15	84,000	28,000	168,000	56,000	194,000	64,667	222,000	74,000	
1½	4¾	3.55	72,000	24,000	144,000	48,000	168,000	56,000	192,000	64,000	
1⅜	4¼	3.00	62,000	20,667	124,000	41,333	144,000	48,000	164,000	54,667	
1¼	4	2.45	50,000	16,667	100,000	33,333	116,000	38,667	134,000	44,667	
1⅛	3½	2.00	42,000	14,000	84,000	28,000	98,000	32,667	112,000	37,333	
1	3	1.58	34,000	11,333	68,000	22,667	78,000	26,000	88,000	29,333	
⅞	2¾	1.20	26,000	8,667	52,000	17,333	60,000	20,000	68,000	22,667	
¾	2¼	0.89	19,400	6,467	38,800	12,933	44,000	14,667	50,000	16,667	
⅝	2	0.62	13,600	4,533	27,200	9,067	31,600	10,533	36,000	12,000	
9/16	1¾	0.50	11,000	3,667	22,000	7,333	25,400	8,467	29,000	9,667	
½	1½	0.39	8,800	2,933	17,600	5,867	20,200	6,733	22,800	7,600	
7/16	1¼	0.30	6,800	2,267	13,600	4,533	15,600	5,200	17,700	5,900	
⅜	1⅛	0.22	5,000	1,667	10,000	3,333	11,560	3,853	13,100	4,367	
5/16	1	0.15	3,400	1,133	6,800	2,267	8,100	2,700	....	....	....
¼	¾	0.10	2,400	800	4,800	1,600	5,400	1,800	....	....	....
Tensile strength of wire per sq. in.			75,000 to 90,000 lbs.		150,000 to 200,000 lbs.		190,000 to 225,000 lbs.		225,000 to 275,000 lbs.		

TABLE LXIV.

Bending Stresses 19-Wire Rope.

Diam. of Bend.	6	8	10	12	14	16	18	20	22	24	26
Diam. of Rope.											
¼	1,801	1,390	1,131	965	827	726	654	586	535	495	455
5/16	3,308	2,568	2,098	1,774	1,536	1,355	1,212	1,096	1,000	920	852
⅜	.....	3,776	3,094	2,620	2,273	2,006	1,796	1,626	1,485	1,366	1,265
7/16	.....	.....	5,351	4,546	3,951	3,494	3,132	2,838	2,594	2,389	2,214
½	.....	.....	.....	6,609	5,755	5,096	4,573	4,147	3,793	3,495	3,241
9/16	.....	.....	.....	.....	8,337	7,393	6,642	6,029	5,519	5,089	4,721
⅝	.....	.....	.....	.....	11,565	10,270	9,237	8,392	7,689	7,095	6,586
11/16	.....	.....	.....	.....	.....	13,360	12,027	10,936	10,027	9,257	8,597
¾	.....	.....	.....	.....	.....	.....	15,309	13,932	12,782	11,807	10,971
⅞	.....	.....	.....	.....	.....	.....	.....	21,403	19,662	18,183	16,910
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	27,612	25,707
1⅛	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	35,620

TABLE LXIV.—*Continued.*

Diam. of Bend.	28	30	36	48	60	72	84	96	108	120
Diam. of Rope.										
$\frac{1}{4}$	423	398	338	250	200	167	144	126	112	101
$\frac{5}{16}$	795	742	621	468	376	314	270	236	210	189
$\frac{3}{8}$	1,178	1,102	924	698	561	469	403	353	314	283
$\frac{7}{16}$	2,063	1,931	1,620	1,226	986	824	708	621	553	498
$\frac{1}{2}$	3,021	2,829	2,376	1,800	1,448	1,212	1,042	913	813	733
$\frac{9}{16}$	4,403	4,125	3,468	2,630	2,118	1,773	1,525	1,338	1,191	1,074
$\frac{5}{8}$	6,145	5,759	4,847	3,680	2,967	2,485	2,135	1,876	1,671	1,506
$\frac{11}{16}$	8,024	7,524	6,201	4,818	3,886	3,257	2,802	2,459	2,191	1,976
$\frac{3}{4}$	10,245	9,609	8,101	6,165	4,977	4,173	3,591	3,153	2,809	2,534
$\frac{7}{8}$	15,805	14,835	12,528	9,556	7,724	6,481	5,583	4,886	4,371	3,943
1	24,047	22,589	19,113	14,614	11,830	9,937	8,566	7,528	6,714	6,059
$1\frac{1}{8}$	33,347	31,347	26,566	20,357	16,500	13,872	11,966	10,523	9,387	8,474
$1\frac{1}{4}$	......	42,036	35,683	27,400	22,239	18,713	16,153	14,209	12,682	11,452
$1\frac{3}{8}$	......	......	48,109	37,028	30,096	25,350	21,897	19,272	17,209	15,545
$1\frac{1}{2}$	......	......	61,238	47,229	38,436	32,403	28,008	24,662	22,030	19,906
$1\frac{5}{8}$	......	......	......	59,094	48,152	40,629	35,140	30,957	27,664	25,005
$1\frac{3}{4}$	......	......	......	74,565	60,844	49,919	44,476	39,203	35,048	31,689
$1\frac{7}{8}$	......	......	......	90,325	73,795	62,379	54,022	47,639	42,606	38,534
2	......	......	......	......	88,409	74,795	64,814	57,183	51,160	46,285
$2\frac{1}{4}$	......	......	......	......	125,387	106,265	92,203	81,428	72,908	66,002
$2\frac{1}{2}$	......	......	......	......	....	145,246	126,185	111,546	99,951	90,540

*Tables LXIII and LXIV were calculated by Mr. William Hewitt, and are published here by his permission. The original, with other data on wire ropes, appeared in a pamphlet entitled "Wire Rope and its Application to Power Transmission," 1901, issued by Trenton Iron Company, Trenton, N. J., from whom copies can be obtained.

FRICTION AND BEARINGS.

The coefficients of friction are designated by f, and to get the horse power required to drive a shaft against this friction, we have:

$$H = \frac{4112\, f\, W\, d\, n}{33{,}000}$$

where W is the total weight on the journal in pounds; d the diam. of journal in inches, n the revolutions per minute.

A journal carries 10 feet of 6-inch shaft and a mortise gear weighing 2400 pounds. The shaft runs at 150 r.p.m. and weighs 900 pounds, therefore $W = 3300$ pounds and the horse power lost is

$$H = \frac{4112 \text{x} .2 \text{x} 3300 \text{x} 150}{33{,}000} = 1\tfrac{1}{4} \text{ h.p.}$$

f is here taken for a mineral oil and 16 pounds pressure per square

inch, as .2. In addition to the above weights there may be the equivalent weight of the thrust due to the driving gear In the above example suppose 500 h.p. is transmitted. The driving wheel is 6 feet in diameter and the pinion 4 feet. There will therefore be

$$\frac{500 \text{x} 33{,}000}{\text{velocity of tooth in feet per min.}}$$

pounds pressure against the bearing due to this thrust or 8760 pounds. Adding this to the weight of shaft and gear, we have

$$\frac{4112 \text{x} .2 \text{x} 12060 \text{x} 150}{33{,}000} = 4.5 \text{ h.p. lost}$$

or about 1 per cent.

Thurston gives the following coefficients of friction.

TABLE LXV.

VALUES OF f.

Oils.	Pressures.			
	8 lb. per sq. in.	16 lb. per sq. in.	32 lb. per sq. in.	48 lb. per sq. in.
Sperm, lard, neat's foot, etc.	.159 to .25	.138 to .192	.086 to .141	.077 to .144
Olive, cotton-seed, rape, etc.	.16 to .283	.107 to .245	.101 to .168	.079 to .131
Cod and Menhaden.........	.248 to .278	.124 to .167	.097 to .102	.081 to .122
Mineral oils	.154 to .261	.145 to .233	.086 to .178	.094 to .222

Always place a bearing *each* side of a gear. Allow no overhung gear.

The standard size of shafting is given in inches, halves, and quarters of an inch, but in reality they are 1-16-inch smaller than their listed size. Thus a 5-inch shaft is actually 4 15-16 in diameter.

For line shafts driving from vertical turbines at ordinary speed the length of the journal should be four times the diameter of shaft. Where necessary three times the diameter will do.

Thurston gives the following list of proper lubricants :

Low temperature as in rock drills driven by compressed air.	Light mineral lubricating oils.
Very great pressures, low speed	Graphite, soapstone and other solid substances.

Heavy pressures, low speed.	The above, and lard, tallow and other greases.
Heavy pressures, high speed.	Sperm-oil and heavy mineral oils, castor oil.
Light pressures and high speed.	Sperm, refined petroleum, olive rape, and cotton-seed oil.
Ordinary machinery.	Lard, oil, tallow oil, heavy mineral oils, heavy vegetable oils.

Practically all bearings must be babited. Different work requires different grades of babit. For the bearings on turbine vertical and horizontal shafts hard babit is usually the best.

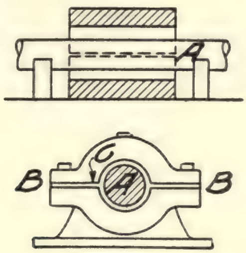


FIG. 443.

To babit the boxes, take the two halves of the box off the bridge-tree and bolt together with a piece of thin cardboard *B* between and center a piece of shaft *A*, Fig. 443. This shaft must be a full $\frac{1}{4}$-inch smaller than the regular shaft. With clay or cement, close up the ends and form at one end a funnel. Heat the whole bearing up so that it is so hot you cannot hold your hand upon it. Heat the babit in a ladle and pour quickly. Now take the halves apart, and with a pean, hammer the babit evenly over its surface to compress it somewhat. Then scrape the edges at *C* so that the bearing will come solidly together. Bolt the halves together and place in a lathe and bore out the bearing to fit the shaft. Such a bearing will wear much longer and run cooler than the usual cheaply made affair. Fig. 444 shows a bearing

of good design. It has a thrust collar and an oiling ring, also dust-proof ends which are filled with carded wool.

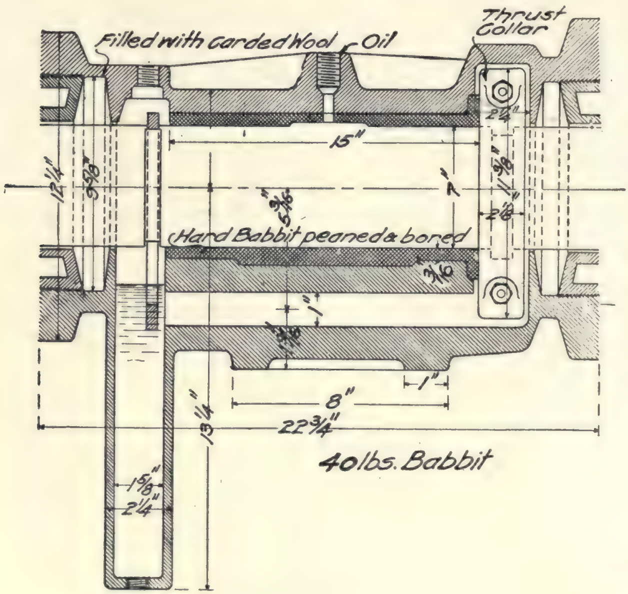


FIG. 444.

HIGH TENSION ELECTRIC TRANSMISSION.

In high tension work it becomes exceedingly important to take the best possible care of the line from the time it leaves the

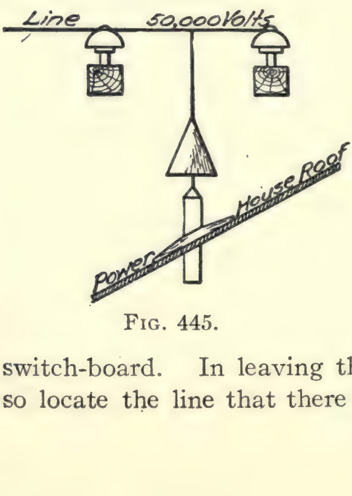


FIG. 445.

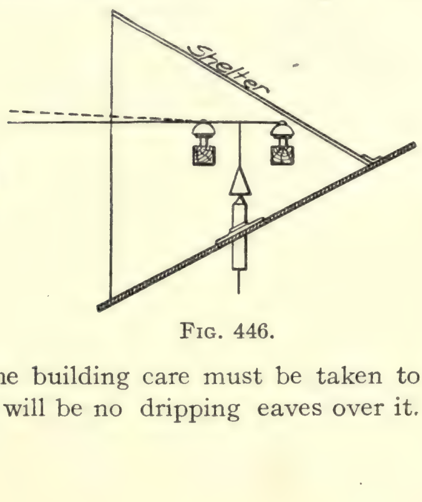


FIG. 446.

switch-board. In leaving the building care must be taken to so locate the line that there will be no dripping eaves over it.

One method is to take the line out through the roof, as in Fig. 445. This was the plan adopted for the Missouri River 50,000-volt transmission plant. It has the defect of being open to snow drifts and sleet. A roof added as in Fig. 446 would serve as a great protection.

POLES.

The best practice to-day is to have two separate and distinct pole lines wherever the continuous operation of the plant is considered of great importance. The Missouri River transmission line is 65 miles long and has two pole lines the whole distance. The pole lines should be far enough apart so that one line could not possibly fall across the other. Each line must have a cleared path wide enough so that no tree or limb can fall upon it. Highways should be avoided where a private right of way can be procured, on account of the danger of the insulators being thrown at or shot at by passing boys and nimrods. Wires passing houses and play-grounds are constantly being crossed by kite strings, etc., and where the line has to pass such places 65-foot poles should be used.

The proper height of the poles will depend entirely on the topography of the country, but generally speaking, outside of towns and in a clear field 35-foot poles should suffice. In Fig. 447 is shown a pole top used on the 50,000-volt transmission referred to above, Fig. 448 being a section of the glass insulator. The pin is of oak boiled in paraffin, and it was found that the pin alone would stand a pressure of 50,000 volts. The glass sleeve is to keep the pin dry.

Poles carrying smaller wires than the above, say Nos. 6 to 2, could be placed 125 feet apart. There is a growing tendency to place the poles farther apart and make each pole a more perfect insulating medium and approaching more nearly the tower. There are now a number of plants transmitting over steel towers placed several hundred feet apart.

All wooden poles set in earth should be of good, sound, well-shaped, live cedar wood, not less than 7 or 8 inches in diameter at the top. They should be cut square at both ends and stripped of their bark. All knots should be trimmed off and no pole should have more than one curve in it.

Standard specifications for cedar poles, with 5-inch tops

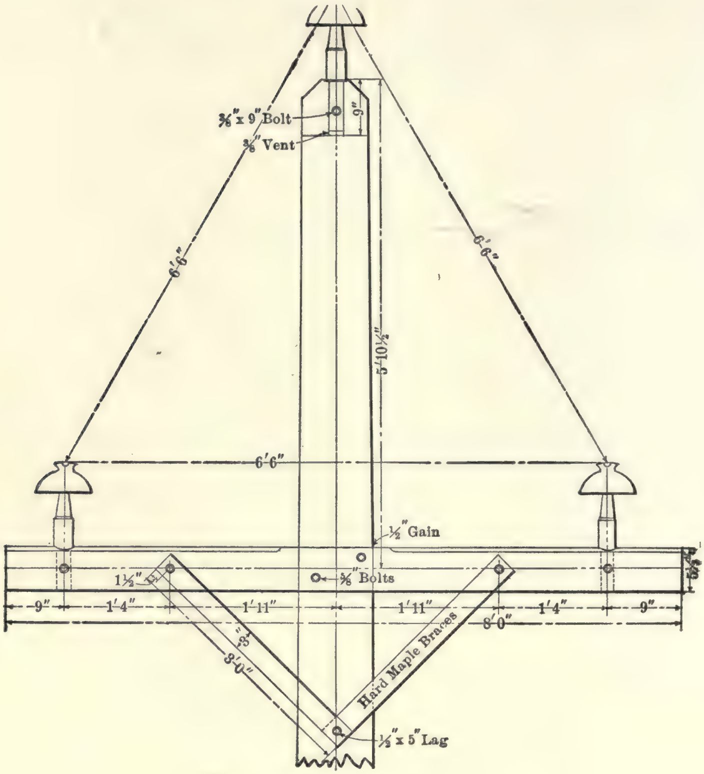


FIG. 447

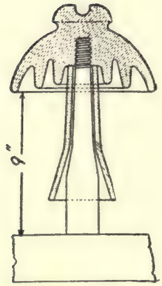


FIG. 448.

and 25 feet long and upwards, are as follows: All poles must be cut from live growing timber, peeled and reasonably well proportioned for their height. Tops must be reasonably sound and when seasoned must measure as follows: 5-inch tops must measure 15 inches in circumference; 6-inch tops, 18½ inches; and 7-inch tops, 22 inches. If poles are green, fresh-cut or water soaked, the 5-inch tops must be 5 inches plump in diameter; 6-inch tops 19½ in circumference and 8-inch tops 22¾ inches.

TABLE LXVI.

Height of Pole.	Depth of Hole.
35 ft. to 45 ft.	5 feet.
50 " 55 "	6 "
60 " 80 "	7 "

One way sweep allowable not exceeding one inch for every 5 feet. The part of the pole in the ground is not included in measurements for sweep. Butt rot in the center, including small ring rot outside the center must not exceed 10 per cent. the area of the butt. Butt rot which plainly seriously impairs the strength of the pole above ground is a defect. Wind shake is not a defect unless very unsightly. Rough, large knots, if sound and trimmed smooth are not a defect.

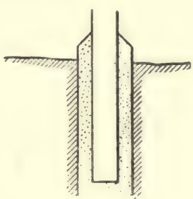


FIG. 449.

The depth a pole should be set in the ground depends on many conditions, such as character of soil, weight of the wires supported, exposure to heavy winds, etc., but roughly, Table LXVI will give the depths which meet average conditions. The part of the pole in the ground is, of course, the first to decay, and though many methods have been adopted to preserve the wood, there is no definite knowledge on the subject yet. Hot tar and

carbolineum are sometimes used. A good plan to preserve the poles from decay and also to protect them from grass fires is shown in Fig. 449. The hole is dug about 12 inches larger in diameter than the pole, the pole set in place and concrete rammed in. The concrete is mixed in a wagon fixed up especially for the purpose (see Fig. 450); and the holes all having been dug a gang of men go ahead and have each pole ready by the time the concrete wagon gets to it. This plan adds about \$1.50 to \$2 to the cost of the pole, but more than doubles the life. The

FIG. 450.

holes may be a foot shallower than given in Table LXVI, as the area of the concrete is greater than that of the bare pole.

The strain on one pole carrying a heavy transmission line and tending to break it off at right angles to the line, may amount to as much as 4000 pounds, applied at top end, though this would only be the case where the wind amounted to a gale; 2000 pounds is usually taken.

Cedar is rapidly becoming exhausted, and within the last five years has almost doubled in price. It therefore becomes important to find a substitute. Oak is stouter than cedar, but

rots in the ground. The setting shown in Fig. 449 protects the pole from rot, and should permit the use of less durable woods. Fig. 451 shows a pole which depends for its lateral support upon three side guys of galvanized strand cable rather than upon the earth. Such a setting makes the installation of poles in rock an easy matter, as no holes have to be blasted. It is especially adapted to boggy or sandy ground, and serves to protect the poles from prairie fires, etc.

Poles should not be guyed to trees or buildings.

The average life of poles when set in the ground is given below:

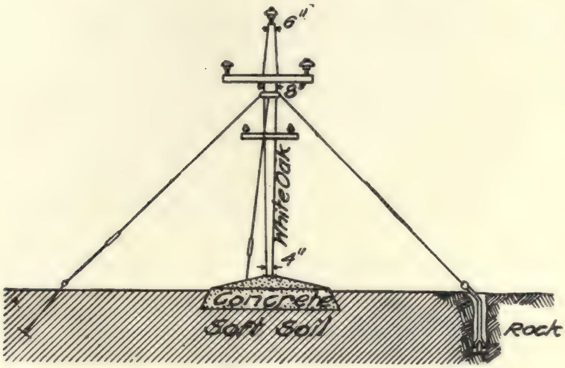


FIG. 451.

Norway pine...............	7 years
Chestnut..................	15 "
Cypress...................	13 "
White cedar...............	10 "

By setting up longer poles than actually necessary at the start the decayed ends may be cut off and the reduced pole set in a new hole, thus doubling the life. The earth around the pole must be thoroughly tamped.

The number of men required to set up a pole is given in Table LXVII though the number given may be somewhat reduced in the case of poles 60 feet and over in length by having a good derrick mounted on a wagon especially fitted up for the purpose.

Climbing spikes are made of 9/16 inch square iron 9 inches long and one man can drive about 200 to 250 per day.

TABLE LXVII.

LABOR REQUIRED TO SET POLES.

Height of Pole.	No. men required.	No. of poles set per day including digging
30 feet	6 with pikes	18
40 "	7 " "	13
60 "	10 " derrick	
70 "	12 " "	
80 "	15 " "	

This is the age of concrete-steel construction and in no way is its application shown to better advantage than in concrete-steel poles. Fig. 452 shows a 35 foot concrete-steel pole; at the top is cored a hole D to receive the top pin. The gain for the

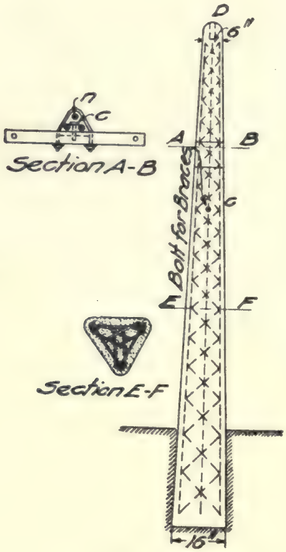


FIG. 452.

cross arm is one inch deep and the arm is bolted as shown in detail A B. The bolt is passed around the pole rather than through it to avoid touching the bars n. The reinforcing as here shown consists of 1″x3″ Kahn bars, but any other reinforcing will make a good pole.

A 35-foot pole costs about $20, depending on the cost of the concrete, about ½ yard being required. A 45-foot pole will cost about $45.

Such a pole will stand a greater transverse strain than will cedar pole and will last indefinitely; 2000 pounds horizontal pull at the top is usually allowed for ıarge poles.

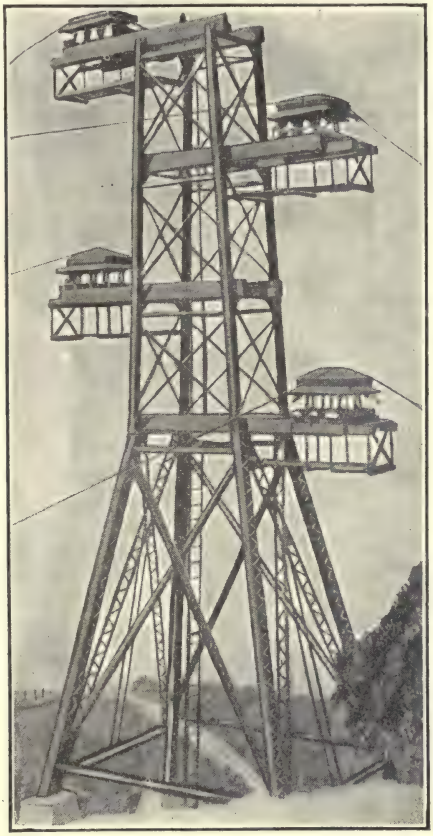


Fig. 453.

Frequently rivers, ravines or even bays have to be crossed with the transmission line, in which case the pole becomes a tower. The tower shown in Figs. 453 was built to carry four ⅞-inch cables over a span of 6200 feet. The tower shown

is 65 feet high and the one at the other end is 225 feet high. Fig. 454 shows one of the four saddles carried on each tower.

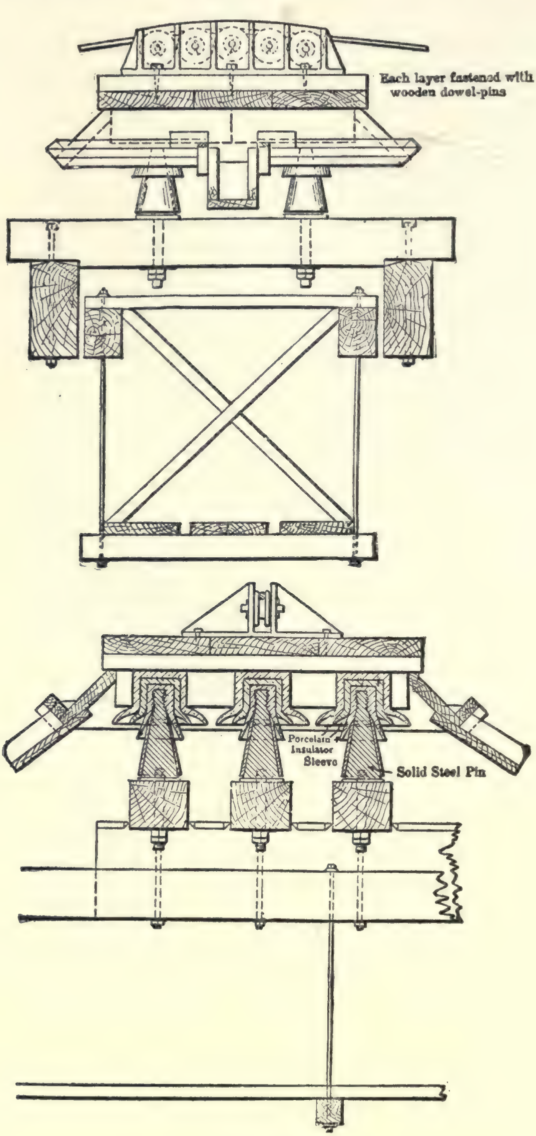


Fig. 454.

Fig. 455 shows the insulating link connecting the anchor and the cable, there being two connected to each cable.

The oil shown in Fig. 455 is for the purpose of keeping the micanite in a good state for insulation, it having been found to deteriorate.

A span of 500 feet over a river was successfully accomplished by bringing the wires, No. 1, a complete turn over an 8x10-inch timber, properly supported as in Fig. 456. Each of the four anchors is built on suspension bridge principles and resists a pull of 12 tons. The cables have a conductivity equal to No.

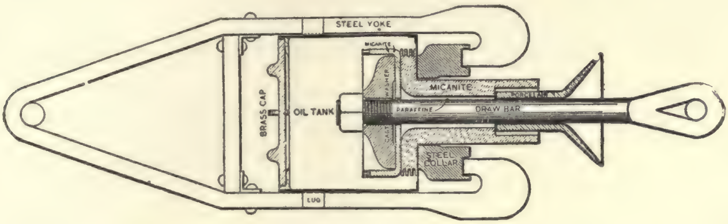


FIG. 455.

2 copper, weigh 7080 pounds each, and have a sag 100 feet and have a breaking stress of 98,000 pounds. The voltage used is 60,000.

The same principle may be carried out for smaller spans, building the towers of timber and using less massive insulators.

On account of wind pressure and added strain due to the

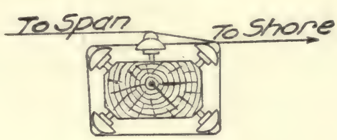


FIG. 456.

falling of one or more poles, it is necessary to thoroughly brace up the line. Also at every turn of the line the corner pole must be well braced. On straight transmission line work every eighth pole should be thoroughly guyed. The rule is to allow 50 poles to the mile, the extra 6 or 10 poles serving for bracing and anchors. If the poles are of the type shown in Fig. 451 this will, of course, be unnecessary.

At the end of a line the poles should be guyed as in Fig. 457, slack being left in the proportion shown, so that only a part of

the strain comes on any one of the poles. The dead men *A* (see Fig. 457) are usually slanted in the direction of the guy unless the soil is very hard.

There are several patent anchors on the market, which are simply driven into the ground and expanded in place. A post hole auger may be used for sinking anchors.

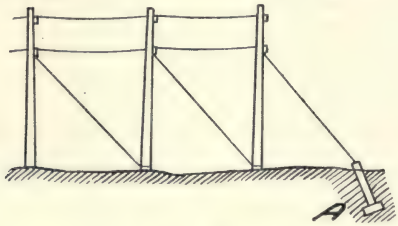


FIG. 457.

CROSS ARMS.

Cross arms may be of yellow pine, white oak, creosoted white pine, chestnut or any other first-class wood.

For ordinary lighting lines the dimensions vary from 30 inches long, $3\frac{1}{4}$x$4\frac{1}{4}$-inches section, for 2-4-pin arms to 8 feet long, $3\frac{3}{4}$x$4\frac{3}{4}$ inches to 4x5-inches section, for 6-8-pin arms.

They are fastened to the pole by means of two $\frac{1}{2}$-inch to $\frac{5}{8}$-inch bolts or lag screws and gained into the pole $\frac{1}{2}$-inch.

All arms of more than 4 pins must be braced with flat iron braces not less than $1\frac{1}{4}''$x$\frac{1}{4}$x$27''$ bolted to the cross arm with $\frac{3}{8}$-inch carriage bolts and the two ends fastened to the pole with $4''$x$\frac{3}{8}''$ lag screw.

On very high tension transmission lines these braces are sometimes made of wood, see Fig. 447.

A general practice on transmission lines is to give the arms a thorough dipping in hot asphaltum compounds, sometimes a coating $\frac{1}{8}$-inch thick being obtained. This is applied after the wood has been well-seasoned and helps to prevent a breakdown should an insulator give out. Boiling the arms in linseed oil is also a good plan.

Cross arms should be placed so as to alternately face each other on adjacent poles and be back to back on the next two. This is to add safety to the bolts should several wires break or a pole fall.

Double cross arms, that, is one on each side of the pole must be placed at all abrupt changes in direction and on all end poles.

PINS.

Pins should be made of the very best material and are made of oak or locust or iron.

If oak is used it must be well boiled in paraffin or linseed oil else they will decay inside of six or eight years. On the Missouri River transmission line, see Fig. 448, the pin is depended on to a certain extent for insulation.

All pins should be bolted into the cross arm as in Fig. 447

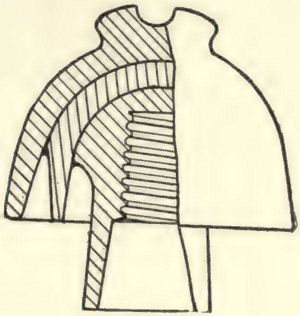


FIG. 458.

especially when the line passes over very hilly country, as otherwise a pole may be placed between two more lofty ones in which case there is a tendency to pull the pin out of its socket. If nailed, the moisture which collects around the pin will soon rust the nail off. The cost of the pin shown in Fig. 448 is about 25 cents.

INSULATORS.

In selecting the proper insulators it is sometimes hard to choose between glass and porcelain. Glass is transparent, which enables internal defects to be detected and renders the cavities undesirable tenements for insects. It does not present a very good mark to the small boy and lunatic. It is cheaper than porcelain. On the other hand porcelain may now be obtained in dull colors which do not attract the eye. They

may be chipped by bullets or stones without being entirely disabled. It is stronger than glass and less hydroscopic. However, in actual practice both glass and porcelain have been used with good success.

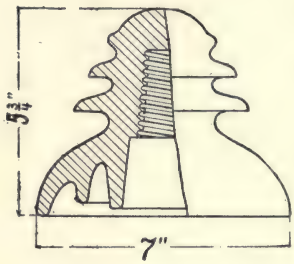


Fig. 459.

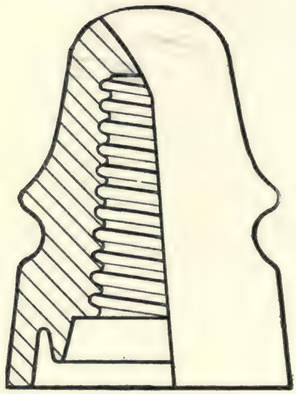


Fig. 460.

On the 40,000 volt Provo transmission line, a glass insulator is used over 105 miles of line, see Fig. 459.

The Edison Electric Company's 83 mile, 23,000 volt-line uses a porcelain insulator of about the same size as the glass one used in the Provo system, and there seems to be much less

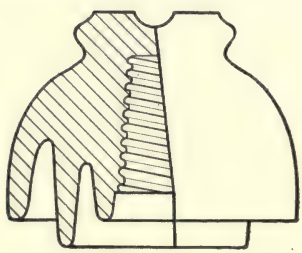


Fig. 461.

leakage of current. Much may be said for and against either glass or porcelain, but in view of the increased experience in porcelain manufacturing and the superior satisfaction now procured it is the author's opinion that porcelain is much the better for high tension work.

Porcelain insulators are made white or chocolate colored, the latter being preferred on account of being less conspicuous.

For all ordinary pressures up to 2000 volts it might be well, on account of cheapness, to use a glass double petticoat insulator, something like Fig. 460. These cost $40 per thousand.

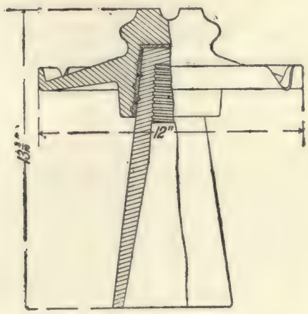


FIG. 462.

For potentials between 2000 and 5000, a glass insulator having more petticoats as in Fig. 461, costing about $55 per thousand should be used.

For higher voltages, all things considered, porcelain should

FIG. 463.

be used. These larger porcelain insulators are made in two or more parts so as to avoid large pieces of porcelain, which could not be vitrified properly. These parts are fastened together by means of sulphur (Fig. 458). Frequently a combina-

tion of glass and porcelain is used as in Fig. 462, made by the Locke Company. A porcelain insulator about 10 inches high and 9 inches in diameter, good for 30,000 to 40,000 volts, costs $500 to $750 per thousand.

The R. Thomas & Company of East Liverpool, O., make a good line of insulators, one of which is shown in Fig. 463. This particular insulator was tested to 120,000 volts, it weighs 23 pounds and costs about $2500 per thousand.

The common rule is to test the insulator with twice its work-

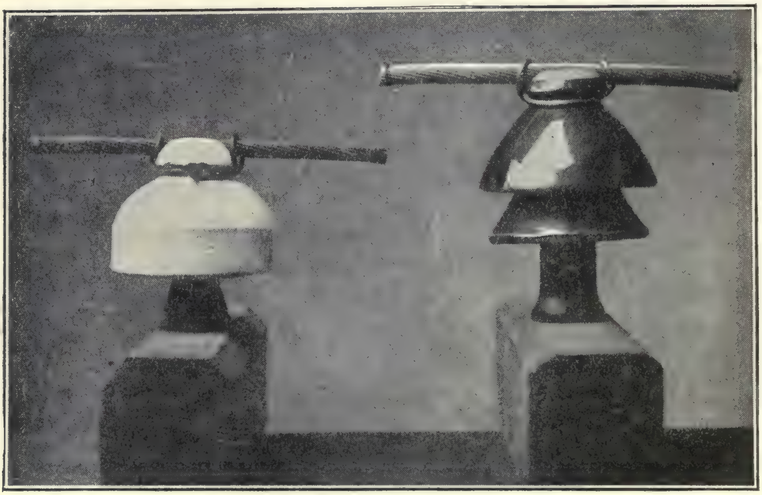


FIG. 464.

ing voltage, that is, it is given a factor of safety of 2, which is, for a three-phase line, equivalent to a factor of 3 between the wire and earth.

It is a good plan to adapt the insulator to the conditions peculiar to the location even if a special design is required. The makers of the best insulators announce that they are ready to make insulators from designs furnished by the engineers.

In Fig. 464 is shown the Niagara type used for a 11,000-volt transmission; the one at the left is the old one and the one at the right is the new one.

THE LINE.

Transmission lines are frequently made of *medium* hard drawn copper or aluminium, though a large proportion are of *hard-drawn* copper. This latter has a breaking tensile strength of from 60,000 to 70,000 pounds per square inch, and while the conductivity is from 2 to 4 per cent. less than the annealed, and somewhat difficult to string up without injury, it is to be preferred. If the wires are larger than No. 1 or 0, it is best to use a stranded hard-drawn cable as it will be more flexible and is especially adapted to alternating current work.

Aluminium has not been generally adopted yet, though there are some very important transmissions using it. It at first proved to be unreliable and is still difficult to solder. It has some very valuable features, however. It is cheaper than copper for the same conductivity and is 1.3 times larger in diameter than copper, and half as heavy.

Its tensile strength should not be taken at more than 15,000 to 17,000 pounds per square inch. This increase in size of aluminium wire over copper would be a disadvantage were it not for the fact that the larger the wires the less the leakage between them.

Were it not for the low tensile strength and added area exposed to wind pressure, the poles could be separated more for aluminium than for copper, but owing to these facts the same spacing is used in both cases. However, the strain is much less on the insulators.

All aluminium transmission wires must be stranded to insure against breaking. All fastenings must be especially solid to prevent slipping as it wears rapidly. Wherever aluminium is joined with any other metal as solder, etc., it must be covered with a waterproof covering as otherwise a galvanic action will be set up which would soon destroy the joint. The McIntyre joint is one of the best for this purpose. It consists of a tube slipped over the wires and then twisted together by means of a special tool.

For long transmission allowance has to be made for the added electric capacity of aluminium wires, being about 5 per cent. more than for copper.

A smaller copper wire than No. 5 should not be used for transmissions on account of its lack of strength.

On transmission lines the wires are strung in the top of the insulators in which case they are fastened as shown in Fig. 464 However, on the smaller side lines they are tied to the side as in Fig. 465. When a series tap is taken off, the connections are as shown in Fig. 466.

In certain localities the wires have to be insulated with some continuous covering. This consists of an insulating compound and a protective covering. The best insulation has for the first covering some rubber composition and for the outer a

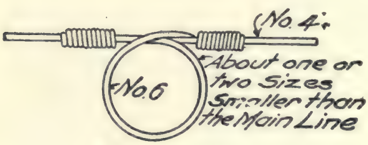


FIG. 465.

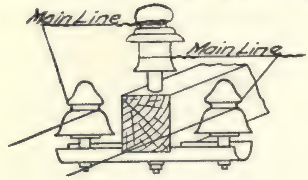


FIG. 466.

hard cotton braid. If the wire is to be continually moist gutta-percha is better than rubber.

The length of wire on a long transmission must be carefully figured as the sag between poles amounts to considerable amount.

The length L (Fig. 467) between two poles A and B is

$$L = H + \frac{8d^2}{3\,H}$$

$$\text{and } d = \frac{H^2\,W}{8\,T}$$

where d is the sag in feet, at middle of span, H the span from A to B in feet, T the tension in pounds in wire at C and W the weight in pounds of wire per foot.

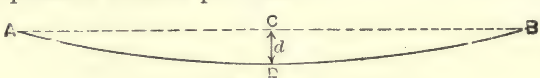


FIG. 467.

Due allowance must be made for changes in temperature, the tension, of course, being greatest in cold weather. From Table LXVIII it will be seen that if a wire is being strung between poles 100 feet apart, when the temperature is 80 degrees Fahrenheit, and give it a sag of $1\frac{3}{4}$ feet, the sag will only be .158 feet When the temperature falls to $-$ 10 degrees Fahrenheit. Then from $T = \dfrac{H^2\,W}{8\,d}$ the tension at that temperature and sag may be found.

In Table LXVIII. the column under the $-10°$ F. gives this sag when the wire is under a tension of 30,000 pounds per square inch, which is only allowing a factor of safety of 2 for medium hard drawn copper. It should be 4 as the tension T is directly proportional to the sag d. In the above case, by doubling the sag, the desired factor of safety will be obtained. Allowance must be made for an added weight of sleet and wind which together may add 20 to 30 pounds to the total weight per wire.

A telephone is more than a luxury to a power station and should be one of the first things provided for. It is also of

TABLE LXVIII.

TEMPERATURE IN DEGREES F.

Span in feet.	−10°	30°	40°	50°	60°	70°	80°	90°	100°
	Deflection in Feet.								
50	.041	.5	.666	.75	.75	.833	.916	.916	1
60	.0583	.666	.833	.916	.916	1.00	1.083	1.083	1.166
70	.0833	.833	.916	1.00	1.083	1.166	1.25	1.25	1.41
80	.100	.916	1.083	1.166	1.25	1.33	1.41	1.5	1.58
90	.133	1.083	1.166	1.33	1.41	1.5	1.58	1.66	1.75
100	.158	1.166	1.33	1.41	1.58	1.66	1.75	1.92	2.00
110	.192	1.33	1.5	1.58	1.75	1.83	2.00	2.08	2.18
120	.233	1.41	1.58	1.75	1.83	2.00	2.18	2.25	2.33
140	.308	1.66	1.92	2.08	2.25	2.33	2.50	2.66	2.75
160	.408	1.92	2.18	2.33	2.50	2.66	2.83	3.00	3.16
180	.516	2.18	2.41	2.66	2.83	3.08	3.25	3.42	3.58
200	.641	2.58	2.75	3.00	3.16	3.42	3.58	3.75	4.00

great value during construction. The telephone line may be strung on the transmission pole line. It will then also serve as a leakage detector, as any great amount of leakage or a cross may be heard in the receiver.

In running the telephone lines on the transmission pole line the transpositions are sometimes made by placing brackets on the poles with both brackets on the same side of the pole, and at transpositions, placing the brackets on opposite sides of the pole, as in Fig. 468, one of the brackets being made long in the sketch simply to indicate the upper bracket, and shows how the upper wire is brought down under the bracket on the pole to the left.

These transpositions should be made about once to the mile on short lines and say six to eight times for the full length on longer ones. A very common way is to provide a two-pin arm for the telephone line and use a double type two-groove insulator at the transpositions. A No. 6 Bell lightning arrester should be installed at each end of the line, though not absolutely necessary.

The greater the distance between the wires the greater the *induction* and therefore the greater the drop in voltage. There are, however, good reasons why the wires should be as far apart as possible. Birds frequently fly into the wires, especially owls; therefore if the wires are several feet apart, the wings of the birds cannot span them. In the West great trouble was experienced from cranes alighting on the wires, so that a distance of from six to seven feet was found necessary between wires.

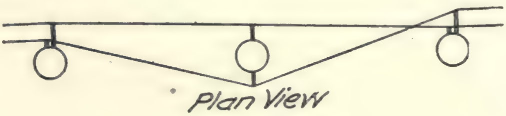


FIG. 468.

Sticks are often thrown across the wires, but if they are far enough apart it would take such a heavy stick that it could not be successfully thrown up to the line. The farther the wires are apart the greater may be the sag and therefore there will be less strain on the wires and insulators. The farther the wires are apart the less will be the leakage across the intervening air.

It is well to calculate the inductive drop or inductive reactance, as it is called, of the line, due to the spacing of the wires, where the distance is unusually great.

The following method of calculating the line losses for alternating currents was given in the *American Electrician* of June, 1897, and as it takes into account the different spacing of the transmission wires, it is given here.

EXAMPLE.—Power to be delivered to the consumer is 250,000 watts; e.m.f. at the consumer's end of the line is 2000 volts; distance of transmission, 10,000 feet; distance between wires is 18 inches.

To start with, assume the size of wire as No. 0. The power factor is .80, and the frequency 60 cycles per second, or 7200 alternations per minute single-phase.

$$\frac{250{,}000}{.8} = 312{,}500 \text{ apparent watts to be delivered.}$$

$$\frac{312{,}500}{2000} = 156.25 \text{ amperes (current in each wire).}$$

From Table LXIX, under the heading 18 inches and corresponding to a No. 0 wire, we find .228. Then we have, $156.25 \times 10 \times .228 = 356.3$ reactance volts. This amounts to 17.8 per cent. of the 2000.

From the column headed resistance volts, we have for a No. 0 wire and 18 inch spacing, .197. Therefore we have $156.25 \times 10 \times .197 = 307.8$ as the resistance volts lost. This is 15.4 per cent. of the 2000 volts.

Now referring to the curves, Fig. 469, we follow up the vertical .8 to the first circle. At this point lay off the resistance volts in a horizontal direction to the right. From the end of this horizontal line thus laid off erect a perpendicular equal to the reactance volts lost. In this case, as shown, the top of the vertical comes nearly to the circle denoting a drop of 24 per cent. Therefore the drop in terms of the generator e.m.f. is

$$\frac{24}{100+24} = 19.3 \text{ per cent.}$$

We have found the resistance volts to be 307.8 and the current to be 156.25 amperes. Hence the power lost 307.8×156.25

$= 48.1$ kw. The per cent. loss is $\dfrac{48.1}{250+48.1} = 16.1$ per cent.

For two and three phase transmissions find the current in the conductors as in the following method (General Electric) and proceed as in the above example.

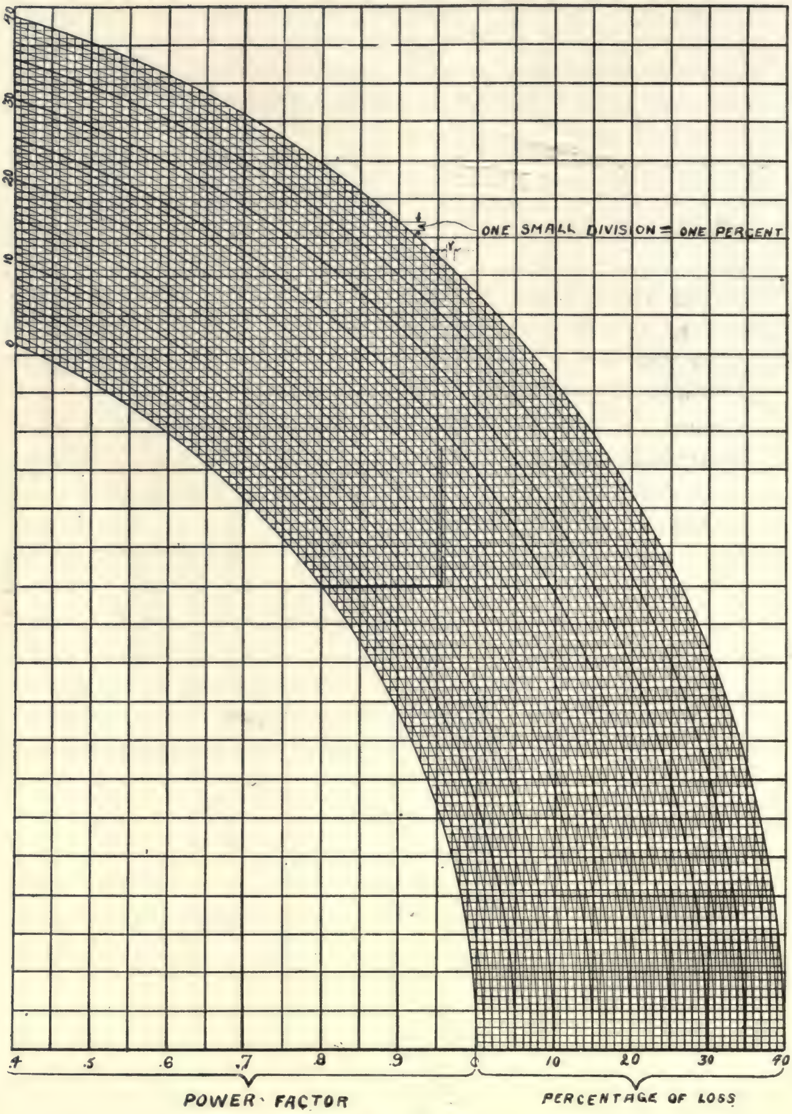

40
30
20
10
0
ONE SMALL DIVISION = ONE PERCENT
.4
.5
.6
.7
.8
.9
0
10
20
30
40
POWER FACTOR
PERCENTAGE OF LOSS

FIG. 469.

TABLE LXIX.

DROP DUE TO INDUCTIVE REACTANCE.

Size of Wire. B. & S.	Weight per 1000 ft. single wire lb.	resistance volts per 1000 ft. pole line. For 1 amp.	Reactance volts per 1000 ft. of pole line for one ampere ($\sqrt{\text{mean square}}$). 7200 cycles per min. for distances between conductors given.										
			½″	1″	2″	3″	6″	9″	12″	18″	24″	30″	36″
0000	639	.098	.046	.079	.111	.130	.161	.180	.193	.212	.225	.235	.244
000	507	.124	.052	.085	.116	.135	.167	.185	.199	.217	.230	.241	.249
00	402	.156	.057	.090	.121	.140	.172	.190	.204	.222	.236	.246	.254
0	319	.197	.063	.095	.127	.145	.177	.196	.209	.228	.241	.251	.259
1	253	.248	.068	.101	.132	.151	.183	.201	.214	.233	.246	.256	.265
2	201	.313	.074	.106	.138	.156	.188	.206	.220	.238	.252	.262	.270
3	159	.394	.079	.112	.143	.162	.193	.212	.225	.244	.257	.267	.275
4	126	.497	.085	.117	.149	.167	.199	.217	.230	.249	.262	.272	.281
5	100	.627	.090	.121	.154	.172	.204	.223	.236	.254	.268	.278	.286
6	79	.791	.095	.127	.158	.178	.209	.228	.241	.260	.272	.283	.291
7	63	.997	.101	.132	.164	.183	.214	.233	.246	.265	.278	.288	.296
8	50	1.260	.106	.138	.169	.188	.220	.238	.252	.270	.284	.293	.302

A few years ago it was thought good engineering to allow a loss of 10 per cent or more in transmissions, but to-day a 5 per cent. loss on full load is the general rule for all but extremely long transmissions.

This small loss is made possible by selecting high voltages an empirical rule being to use 1000 volts per mile approximately. Of course there are often other considerations which enter to influence the selection of voltage.

The wiring formulas here given are about the simplest and most exact of any and are in the form gotten out and used by the General Electric Company. They may be used to determine the size of copper conductors, volts loss in lines, current per conductor, and weight of copper per circuit for any system of electrical distribution.

$$\text{Area of conductor, circular mils} = \frac{D \times W \times C}{P \times E^2}$$

$$\text{Volts loss in lines} = \frac{P \times E \times B}{100}$$

$$\text{Current in main conductors} = \frac{W \times T}{E} = I$$

$$\text{Lbs. copper} = \frac{D^2 \times W \times C \times A}{P \times E^2 \times 1{,}000{,}000}$$

W = Total watts delivered.

D = Distance of transmission (one way) in feet.

P = Loss in line in per cent. of power delivered, that is of W

E = Voltage between main conductors at receiving or consumer's end of circuit.

For continuous current $C = 2160$, $T = 1$, $B = 1$, and $A = 6.04$. Table LXX gives values of the constants C, T, B and A when applied to *a. c.* calculations.

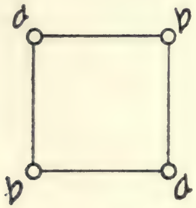


FIG. 470.

FIG. 471.

The formulas are applicable to direct and alternating current systems, and may be used to find the size of wire to transmit any amount of power to any kind of load known to electrical engineering.

In all alternating current transmissions the wires should be transposed at equal intervals, and be placed equidistant from one another, to equalize the inductive drop in the phases. Many transmissions have the wires 36 inches apart, though a greater distance is sometimes to be recommended.

The two circuits of a quarter phase must be arranged as in Fig. 470 or Fig. 471.

Fig. 471 shows the two circuits side by side though they may be above each other or in any position if that position is maintained.

The three wires of a three-phase transmission must be ar-

TABLE LXX.

VALUES OF CONSTANTS IN GENERAL ELECTRIC FORMULAS.

SYSTEM	VALUES OF A.	VALUES OF C. PER CENT. POWER FACTOR.					VALUES OF T. PER CENT. POWER FACTOR.				
		100	95	90	85	80	100	95	90	85	80
Single-phase	6.04	2160	2400	2660	3000	3380	1.00	1.05	1.11	1.17	1.25
Two-phase (four wire)	12.08	1080	1200	1330	1500	1690	.50	.53	.55	.59	.62
Three-phase (three-wire)	9.06	1080	1200	1330	1500	1690	.58	.61	.64	.68	.72

VALUES OF B.

No. of Wire. B. & S. Gauge.	Area Wire. Circular Mils.	Weight of Bare Wire Per 1,000 Ft., Pounds.	Resistance of Wire Per 1000 Feet at 20° C. Ohms.	25 CYCLES. PER CENT. POWER FACTOR.				40 CYCLES. PER CENT. POWER FACTOR.				60 CYCLES. PER CENT. POWER FACTOR.				125 CYCLES. PER CENT. POWER FACTOR.			
				95	90	85	80	95	90	85	80	95	90	85	80	95	90	85	80
0000	211,600	640.73	.04879	1.23	1.29	1.33	1.34	1.52	1.53	1.61	1.67	1.62	1.84	1.99	2.09	2.35	2.86	3.24	3.49
000	167,805	508.12	.06154	1.18	1.22	1.24	1.24	1.40	1.41	1.48	1.51	1.49	1.66	1.77	1.95	2.08	2.48	2.77	2.94
00	133,079	402.97	.07758	1.14	1.16	1.16	1.16	1.25	1.32	1.35	1.37	1.34	1.52	1.60	1.66	1.86	2.18	2.40	2.57
0	105,560	319.00	.09775	1.10	1.11	1.10	1.09	1.19	1.24	1.26	1.26	1.31	1.40	1.46	1.49	1.71	1.96	2.13	2.25
1	83,694	253.43	.1234	1.07	1.07	1.05	1.03	1.14	1.17	1.18	1.17	1.24	1.30	1.34	1.36	1.56	1.75	1.88	1.97
2	66,373	200.98	.1556	1.05	1.04	1.02	1.00	1.11	1.12	1.12	1.10	1.18	1.23	1.25	1.26	1.45	1.60	1.70	1.77
3	52,633	159.38	.1962	1.03	1.02	1.00	1.00	1.07	1.08	1.07	1.05	1.14	1.17	1.18	1.17	1.35	1.46	1.53	1.57
4	41,742	126.40	.2473	1.02	1.00	1.00	1.00	1.05	1.06	1.03	1.00	1.11	1.12	1.11	1.10	1.27	1.35	1.40	1.43
5	33,102	100.23	.3120	1.00	1.00	1.00	1.00	1.03	1.01	1.00	1.00	1.08	1.08	1.06	1.04	1.21	1.27	1.30	1.31
6	26,250	79.49	.3934	1.00	1.00	1.00	1.00	1.02	1.00	1.00	1.00	1.05	1.04	1.02	1.00	1.16	1.20	1.21	1.21
7	20,816	63.03	.4958	1.00	1.00	1.00	1.00	1.01	1.00	1.00	1.00	1.03	1.02	1.00	1.00	1.12	1.14	1.14	1.13
8	16,509	49.99	.6250	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.02	1.00	1.00	1.00	1.09	1.10	1.09	1.07
9	13,090	39.60	.7886	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.06	1.06	1.04	1.02
10	10,382	31.40	.9940	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.04	1.03	1.00	1.00

ranged either as in Fig. 472 or Fig. 473. Fig. 472 is the best and most common method. In Fig. 473 the wires are all on one cross arm and there are two transpositions as shown on the pole line. A quarter-phase three-wire line need not be transposed.

For very long transmissions of a hundred miles or more, special calculations are necessary to find the inductive drop which for a circuit of 100 amperes, at 60 cycles transmitted 200 miles might be 50 per cent.

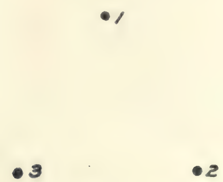


FIG. 472.

FIG. 473.

LIGHTNING PROTECTION.

Lightning is the most persistent foe to the transmission line and must be well guarded against.

The best protection is to provide the arresters and the power-house equipment with choke coils. Choke coils are somewhat expensive but give splendid protection. On a long line the

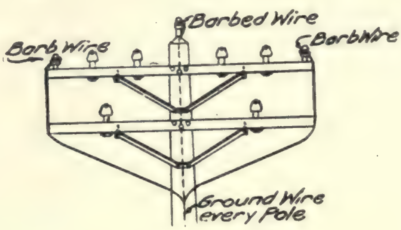


FIG. 474.

arresters should be most numerous near the ends of the line.

The grounds connecting the arresters to earth must be well made else the efficiency of the protection is greatly impaired. A galvanized 1½-inch gas pipe driven five or six feet into moist ground will answer the purpose for the arresters out on the line, but for those protecting the stations connections should be

made to steel penstocks, water pipes, or some such thoroughly grounded device. In the absence of these, a large sheet of galvanized iron should be buried in moist earth and the ground wire well soldered across it. The ground wire must run as directly as possible to the earth plate and any complete turn in it will be fatal to the protection.

Fig. 474 shows a pole line protected by barbed wire such as is used for fencing. This plan has been quite generally adopted in the west; that shown is in Canada. Frequently the barbed wire is fastened to the arm by means of staples, though this is bad practice as the wire breaks at the staple.

Common glass insulators should be used. A liberal sag must be allowed to keep the tension down to a safe figure.

GENERAL REMARKS.

From the foregoing the following observations are drawn:

All pole fittings should be of galvanized iron.

All guy wires should be of at least two No. 8 wires twisted together; for high tension lines the pins and cross arms should be boiled in some sort of oil.

Insulators for medium voltages may be of glass but for high voltages porcelain or a combination of porcelain and glass is best.

The wires should be far apart (from 18 inches to 72 inches) to reduce leakage and make it difficult for a person to throw a stick across them large enough to cause a short circuit, and to lessen the liability of birds causing shorts.

The wires, other things being equal, should be as large as possible, even as large as one inch in diameter to lessen the leakage of energy.

A stranded aluminium wire of large diameter is the best conductor for the purpose if properly strung.

Some form of concrete pole or a pole set in concrete makes the cheapest pole in the long run.

The pole line should be on land controlled by the company, and there should be two distinct transmission lines.

Modern tendency is toward concentrating the points of line insulation and making each pole, or tower, a more perfect insulating medium. One line lately constructed has spans of 700 feet.

All voltages are practicable up to 60,000 or 80,000 volts.

Three-phase is the cheapest system for long distance transmission.

The lower the frequency the more efficient the transmission.

Large wires (aluminum wire is apt to be large) have a certain capacity which on long lines is compensated for by induction coils which are cut out as the load is thrown on.

If the plant is well loaded there will be little trouble from induction or capacity.

A large number of lightly loaded induction motors on the line reduces the power factor with consequent reduction of carrying capacity. A few large synchronous motors run on the same line with the inductive load tend to greatly improve the power factor.

Efficiencies.

There must be some loss in all transmission and transformation of power, and it is quite important to know what per cent. of the theoretical power will be left for sale.

Assuming that there is no loss in the water before it gets into position over the wheels we will first consider the turbine. In Fig. 475 is given the efficiency curves for the more important types of turbine. These are the curves given in the catalogues and we may therefore assume that they represent the best that can be expected. Old turbines tested by the author had an efficiency of 57 per cent. at full load.

For a gate opening of from ¾ to full the maximum efficiency claimed is 85 per cent. With decreased gate the efficiency falls off rapidly on some. The 45-inch Victor under 198-foot head has really the best curve, though its maximum efficiency is only 78 per cent. Few power units are run constantly at full gate, about 60 per cent. being more nearly it; therefore according to the turbine makers own statement, 75 per cent. is all we can expect at the average load and on low heads. On high head 80 per cent. should be the average efficiency.

" In one pair of gears there may be a loss of 10 per cent." (Kent). If the turbine is vertical there must be some means of transmitting the power to the horizontal shaft of the machine. Vertical generators are sometimes employed, but not usually. In most cases gearing must be used. For small turbines a crossed belt or rope may be used when the loss will be about three per cent.

In a line shaft there will be a loss of from two to five per cent. depending on the alignment and character of the oil and bearing metal. 0.5 to 1 per cent. is lost at each bearing on an average.

The next loss will take place in the generator. Fig. 476 shows the efficiency curve of a water wheel type, three-phase generator. This efficiency here shown does not include the excitation, as that is not usually included in the rating of the generator. This loss will usually be about two per cent. The

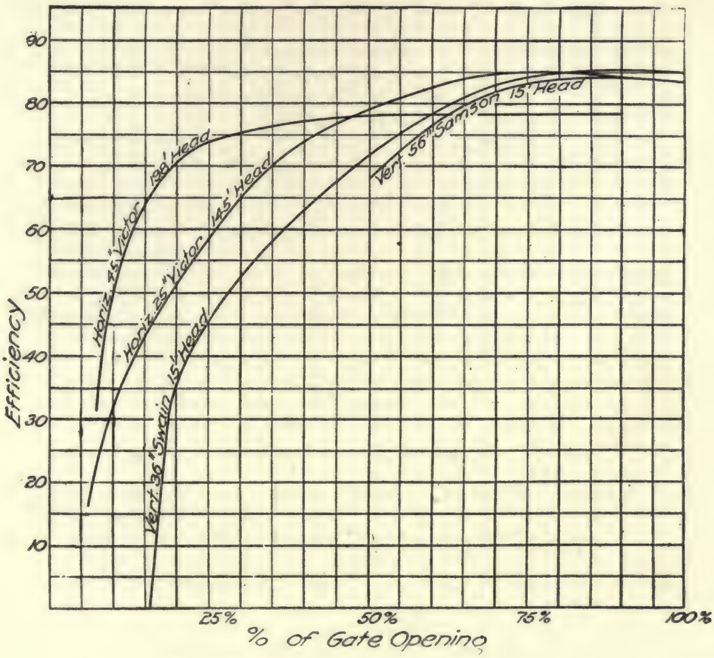


FIG. 475.—Efficiency Curves of Various Turbines,

loss in the two bearings of about one per cent. is included in the curve. The *power* required to drive the exciter is not here considered at all. The capacity of the exciter should be three to five per cent. of the rated capacity of the generator.

The power factor also has much to do with the efficiency. The size or rating at which the generators are sold is for non-inductive load and not an inductive load such as transformers, motors, etc.,

Table LXXI gives the efficiencies for various sizes of core type transformers.

Transformers will carry a peak load of 100 per cent.

The loss in the transmission line, counting that at the switchboard, should not be more than six per cent at full peak load, as a voltage may be selected which will limit it to that amount.

Ordinarily the hydraulic power owner needs to take his power

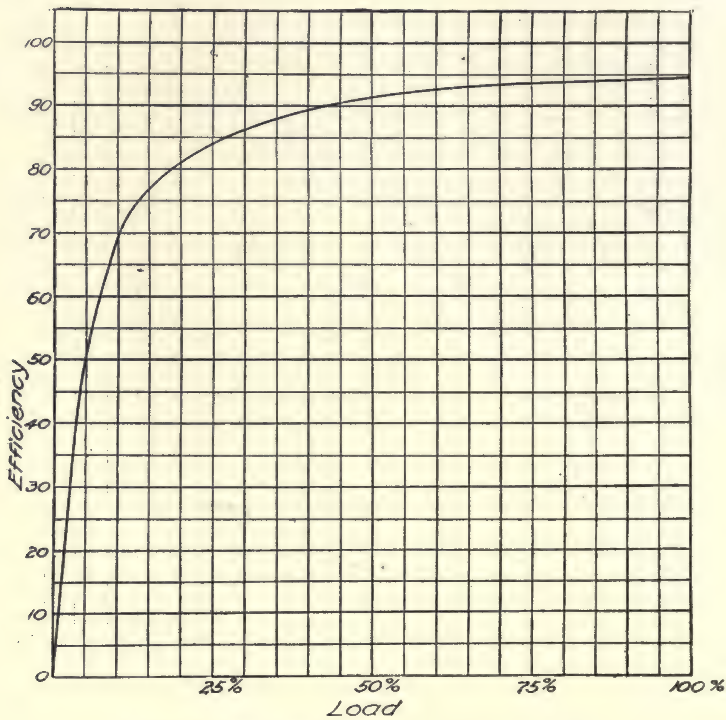


FIG. 476.—The efficiency curve of a generator.

no further than the step down transformer at the end of his line, but frequently he takes a contract to drive a machine or has to transform the alternating current into constant current.

Synchronous motors will not start under full load without the use of friction clutches, and require several times full load current to start, causing a heavy drop on the line.

Alternating current motors of the induction type have a half

load efficiency of 90 per cent. for large motors, 75 to 150 h.p., and 85 per cent. for the smaller sizes. Synchronous motors have an efficiency slightly greater than that of the induction type.

The losses from the switchboard to the trolley of an electric there is what might be called a back pressure set up which reduces the effective capacity of the machine. Thus with a power factor of .8 such as is common with a load of both motors and lights, the capacity of the generator will be only about 80 per cent. of the rated capacity though the *turbine* power required to drive the generator is the *rated* capacity plus the usual 50 per cent. overload, plus 10 per cent. for regulation, plus three to five per cent. for excitation.

TABLE LXXI.

General Electric Type H Oil Transformers.

Crocker.

Watts capacity.	Efficiency.				Weight in pounds.
	Full load.	¾ load.	½ load.	¼ load.	
600	93.5	92.9	91.1	85.2	70
1,500	95.2	95.	94.	90.3	125
2,500	96.	95.9	95.1	92.1	195
4,000	96.4	96.4	95.9	93.6	270
7,500	96.7	96.6	96.2	94.	470
10,000	96.9	96.9	96.4	94.3	535
15,000	97.1	97.1	96.8	95.1	850
25,000	97.3	97.4	97.2	95.9	1210
50,000	97.7	97.7	97.5	96.1	1900

For all probable average loads 90 per cent. is nearly the efficiency we may expect of an alternating current generator, including excitation.

As the generator voltage should not be over 11,000 volts, transformers are used to step up the voltage for transmission.

In selecting the transformers, it is not always best to buy those of highest efficiency unless the additional power lost in the less efficient transformer would be worth more than the difference in cost.

In changing the frequency of an alternating current by means of a synchronous and an induction motor from 12 to 14 per cent. is lost depending on size and running conditions.

To change an alternating current to direct by means of a synchronous converter or motor generator, from four per cent. to eight per cent. is lost.

A loss of about four per cent. is sustained in changing from one phase to another, being the losses in the transformers.

Fig. 477 gives the typical efficiency curves for the various machines used in hydro-electric plants.

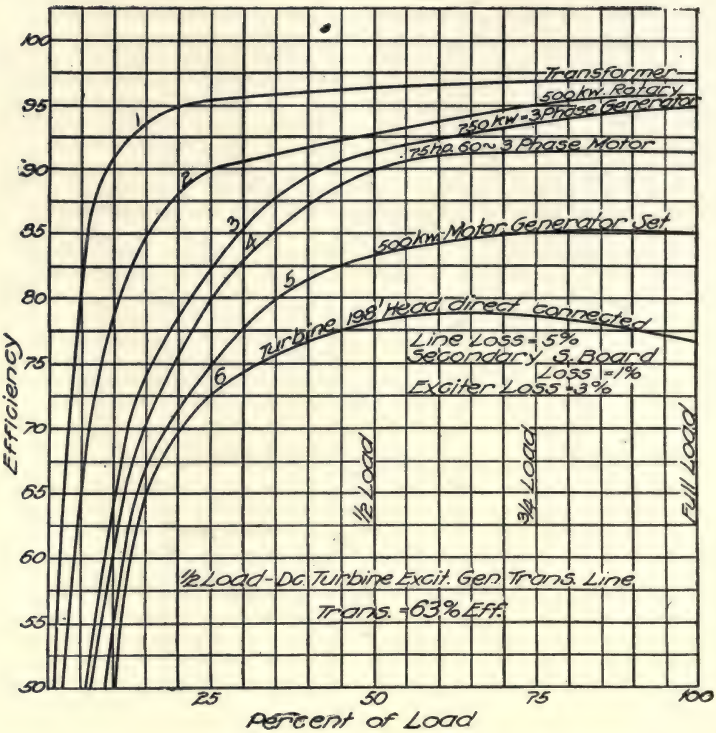


FIG. 477.—Efficiency of various machines at part load.

Fig. 478 shows the successive losses in a transmission system. It starts with 1000 theoretical horse power in the water and takes it through turbine, gearing, generator, step up transformers, substation and synchronous converter, and delivers it to the trolley. "In the motors on the car there will be a further loss of 15 per cent. due to motor gearing, wiring, etc.' (Bell).

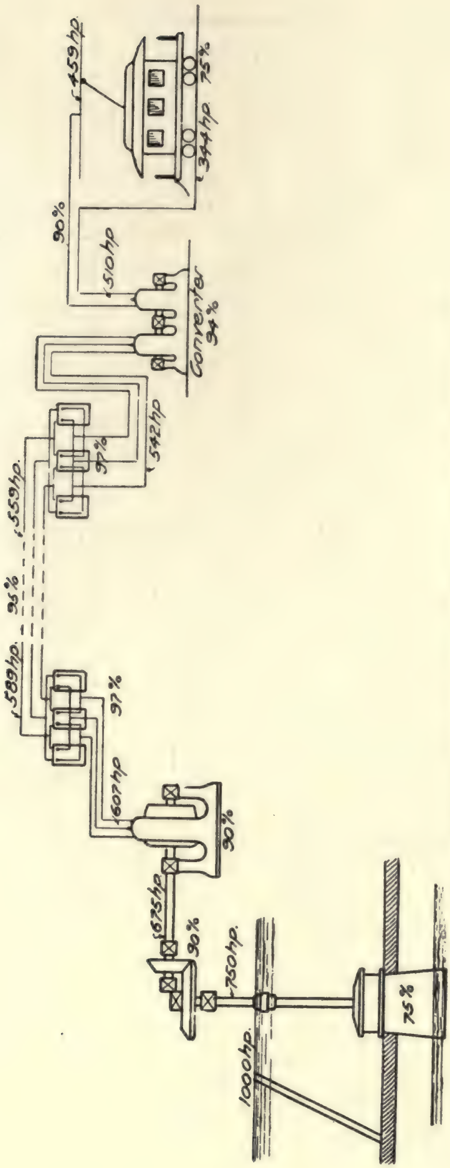


FIG. 478.—Diagram showing losses in typical transmission.

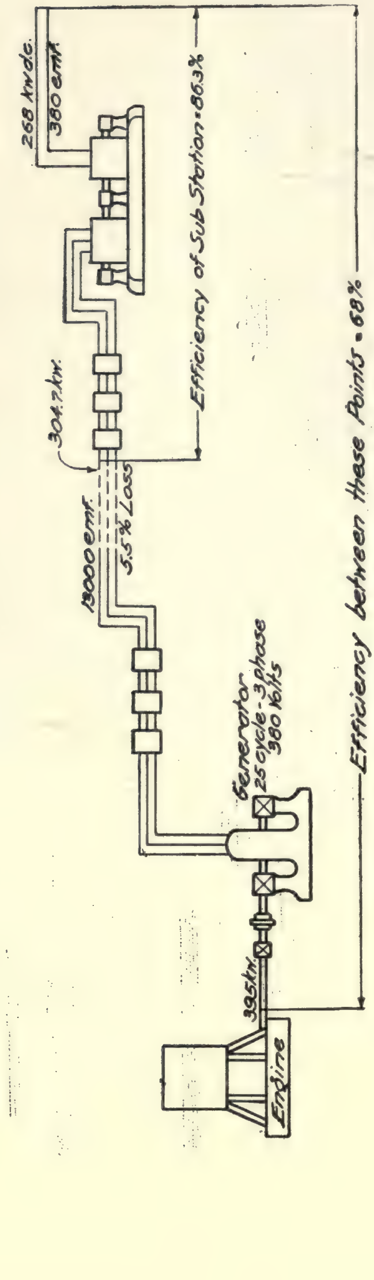


FIG. 479.—Diagram showing losses in an actual transmission.

A reliable test made on the transmission from Lauffen to Frankfort places the efficiency at about third load as follows:

98 h.p. in the water.	78 h.p. at generator Eff: = 79.6 per cent.	66 h.p from generator Eff : = 84.6 per cent.	61 h.p. from step-up transformer. Eff: = 92½ per cent.	58 h.p. to step-down transformer. Eff: = 95 per cent.	53.5 h.p. from-step down transformer. Eff: = 92.2 per cent.

This gives an efficiency of about 50 per cent. from the water to the customer's circuit. The full load efficiency was about 54 per cent. The transmission 109 miles, voltage 12 to 25,000. These losses may seem excessive and yet they are fully as good as can be attained in practice. If all machines could be run at full load the losses could be greatly reduced, but such would not be practice, therefore we get a 51 per cent. efficiency at the end of our line or 510 h.p. Ordinarily this is the amount of power the hydraulic proprietor would have to sell. Meters and other small losses would make this about 500 h.p. or one-half the theoretical power in the water above the dam.

Direct turbine-generator connection would make the loss from turbine to generator 80 per cent., cutting out the gearing loss. Crosby & Bell state that actual tests on existing steam railway plants showed an efficiency from the steam, after generation, to the line of 40 per cent. at Lafayette, Ind., 62.8 per cent.; at Syracuse, N. Y., and another road, 54.6 per cent. This with a line loss of 10 per cent. and car loss of 25 per cent., gives an efficiency of only 40 per cent. at the car; and in the above case would only deliver 204 of the 510 h.p. instead of 240 h.p. as shown. Hence, the hydraulic power plant has the advantage after the sub-station is reached. Fig. 479 shows graphically the results of tests on a new transmission plant.

Efficiency of Old Wheels.

In Fig. 480 is given a set of curves showing the efficiency of old turbines. These curves were made from dynamometer tests on turbines in actual service. Curves 1 to 4 inclusive are by Mr. Wm. O. Webber and given in a paper read before the American Society of Mechanical Engineers, May, 1906. Curve 1 was of a 39-inch Hercules wheel, in good shape and of comparatively late design.

Curve 2 is of a 40-inch Risdon wheel cylinder gate, which had been in use about 10 years.

Curve 3 was a very old cylinder gate, 54-inch Risdon wheel in good condition.

Curve 4, that of a turbine of same make, size and age as 3, but had several nicks in the buckets.

Curve 5 is of a New American wheel 20 years old, tested by Mr. Wm. Kramer.

Curve 6 is of a New American wheel 18 years old, tested by the writer.

These were all vertical turbines and the curves all show the

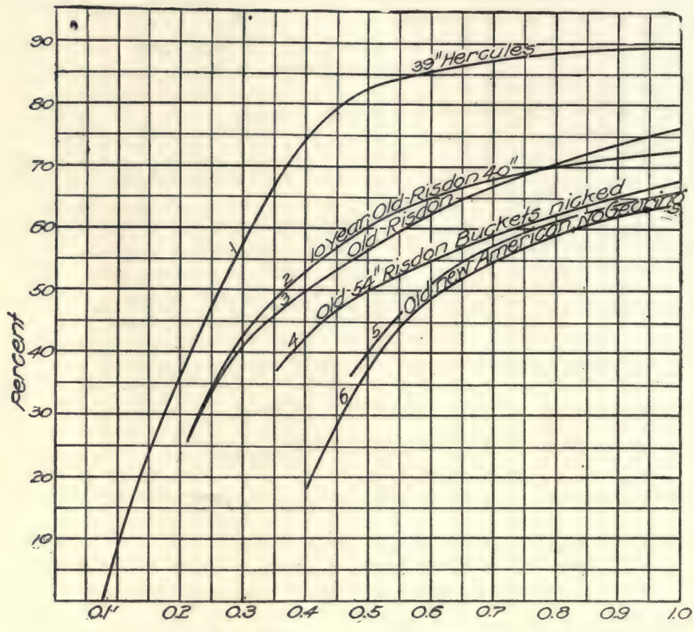


FIG. 480.—Efficiency of old water wheels.

efficiency at end of line shaft except the last, where the dynamometer was placed directly on the turbine shaft.

These curves are extremely valuable to the engineer as showing the relative value of old turbines. It will be seen that the half load efficiency in every case but one falls below 60 per cent. Curve 1 may be considered as being better than the average modern turbine.

Mr. Webber made many tests of efficiency at different speeds, and added emphasis to the fact that the efficiency rapidly falls off when the turbine is run at lower or higher speeds than that for which it is designed.

CHAPTER IX.

TABLES AND FORMULAS.

In this chapter tables and formulas, which it is thought will be useful to the engineer, have been compiled. The tables giving the power and energy in water under different conditions were calculated by the author; the rest of the tables were compiled from various sources, credit being given in each case.

Comparison of Heads of Water in Feet with Pressures in Various Units (Kent).

One Foot of Water at 39.1° F. = 62.425 lb. per sq. ft.
" " " " " " = .4335 lb. per sq. in.
" " " " " " = .0295 atmospheres.
" " " " " " = .8826 in. of mercury at 32°.
" " " " " " = 773.3 ft. of air at 32°.
One pound per sq. ft. at 39.1° F. = 0.1602 ft. of water.
" " " " " " " = 2.307 ft. of water.
" atmosphere at 29.922 in. of mercury = 33.9 ft. of water.
" in. of mercury at 32.1° F. = 1.133 ft. of water.
" foot of average sea water = 1.026 ft. of pure water.
" " of water at 62° F. = 62.355 lb. per sq. ft.
" inch " " " " = .43302 lb. per sq. in.
" pound of water per sq. in. at 62° F. = 2.3094 ft. of water.

Power and Energy Expressions and Their Equivalents.

$$\text{One Watt} = \left\{\begin{array}{l} 1.\ \text{ampere at one volt.} \\ 0.7373\ \text{foot-pounds per second.} \\ 44.238\ \text{foot-pounds per minute.} \\ 2654.28\ \text{foot-pounds per hour.} \\ 0.00134\ \text{horse-power.} \end{array}\right.$$

$$\text{One Kilowatt} = \begin{cases} 737.3 \text{ foot-pounds per second.} \\ 44238. \text{ foot-pounds per minute.} \\ 1.34 \text{ horse-power.} \end{cases}$$

$$\text{One Horse Power} = \begin{cases} 550 \text{ foot-pounds per second.} \\ 33000 \text{ foot-pounds per minute.} \\ 746 \text{ watts.} \\ .746 \text{ kilowatts.} \end{cases}$$

$$\text{One Watt-Hour} = \begin{cases} 2654.28 \text{ foot-pounds.} \\ 1. \text{ ampere-hour at one volt.} \\ .00134 \text{ horse-power hour.} \\ 0.001 \text{ kilowatt-hour.} \end{cases}$$

$$\text{One Horse-Power-Hour} = \begin{cases} 1{,}980{,}000 \text{ foot-pounds.} \\ 746 \text{ watts for one hour.} \\ .746 \text{ kilowatt-hours.} \end{cases}$$

Application of Energy Formulas.

Example 1:—Assume that a given river has a flow of 10,000 cubic feet per minute which is utilized under a head of 11 feet for 10 hours every day, required the energy in horse-power hours which is used per day of 10 hours,

$$\frac{11 \times 10{,}000 \times 10}{528} = 2.083 \text{ horse-power-hours.}$$

On the other hand, suppose that we have a reservoir whose superficial area is 1,500,000 square feet and that we can draw down the surface of this reservoir two feet, that is, we have 3,000,000 cubic feet of water which can be utilized under the average head (allowing one foot for lost head) of 20 feet. Then, substituting in the proper formula, in Table LXXII, we have for 100 per cent. efficiency,

$$\frac{20 \times 3{,}000{,}000}{31{,}600} = 1890 \text{ horse-power-hours.}$$

This represents the total energy available which if used in one hour would be 1890 horse-power; if used in 10 hours would be 189 horse-power.

TABLE LXXII.

Hydraulic Power and Energy Formulas.

Efficiency in per cent.	Power in h.p.	Energy in h.p.-hr. flow.	Energy in h.p.-hr. stored.	Energy in kw.-hr. flow	Energy in kw.-hr. stored.	Energy in acre-ft.-h.p.-hr.	Energy in acre-ft.-kw-hr
100	$\frac{H\,Q_f}{528}$	$\frac{H\,Q_f\,T}{528}$	$\frac{H\,Q_s}{31,600}$	$\frac{H\,Q_f\,T}{707.5}$	$\frac{H\,Q_s}{42,500}$	1.375 H N	1.025 H N
85	$\frac{H\,Q_f}{621}$	$\frac{H\,Q_f\,T}{621}$	$\frac{H\,Q_s}{37,200}$	$\frac{H\,Q_f\,T}{832}$	$\frac{H\,Q_s}{50,000}$	1.169 H N	0.872 H N
80	$\frac{H\,Q_f}{660}$	$\frac{H\,Q_f\,T}{660}$	$\frac{H\,Q_s}{39,500}$	$\frac{H\,Q_f\,T}{885}$	$\frac{H\,Q_s}{53,000}$	1.1 H N	0.82 H N
75	$\frac{H\,Q_f}{704}$	$\frac{H\,Q_f\,T}{704}$	$\frac{H\,Q_s}{42,200}$	$\frac{H\,Q_f\,T}{944}$	$\frac{H\,Q_s}{56,750}$	1.03 H N	0.769 H N
60	$\frac{H\,Q_f}{880}$	$\frac{H\,Q_f\,T}{880}$	$\frac{H\,Q_s}{52,600}$	$\frac{H\,Q_f\,T}{1180}$	$\frac{H\,Q_s}{70,900}$	0.825 H N	0.615 H N

Note.— H = head in feet; Q_f = cu. ft. per min. flow; Q_s = cu. ft. water stored for use; T = time in hours, which power is used; N = number of acres of reservoir area per ft. depth.

The application of the kilowatt-hour formulas is exactly similar to the examples given above.

EXAMPLE 2:—Suppose that we have a reservoir whose superficial area is 12 acres and that we are able to draw down the surface two feet and utilize the water under a head of 20 feet. What is the number of horse-power-hours per acre, assuming an efficiency of 60 per cent. Referring to the proper formula in Table LXXII we have

$$0.825 \times 20 \times 12 = 198 \text{ horse-power hours per acre foot.}$$

Since we are going to use two feet, there will be twice this amount or 396 horse-power hours per acre. If this energy is utilized during 10 hours we would have 39.6 horse-power; if it

TABLE LXXIII.

THE HORSE-POWER IN WATER, FLOWING AT THE RATE OF ONE CUBIC FOOT PER MINUTE AND OPERATING UNDER VARIOUS HEADS AT AN EFFICIENCY OF 85 PER CENT.

Head in feet.	h.p. of 1 cu. ft. of water.	Head.	h.p.	Head.	h.p.	Head.	h.p.	Head.	h.p.
1	.0016088	36	.057953	100	.160980	320	.515136	640	1.030272
2	.0032196	38	.061172	105	.169029	330	.531234	650	1.04637
3	.0048294	40	.06439	110	.177078	340	.547332	660	1.062468
4	.0064392	42	.067612	115	.185127	350	.563430	670	1.078566
5	.008049	44	.070831	120	.193176	360	.579528	680	1.094664
6	.0096588	46	.074051	125	.201225	370	.595626	690	1.110762
7	.0112686	48	.07727	130	.209274	380	.611724	700	1.12686
8	.0128784	50	.08049	135	.217323	390	.627822	710	1.142958
9	.0144882	52	.08371	140	.225372	400	.643920	720	1.159056
10	.016089	54	.086929	145	.233421	410	.660018	730	1.175154
11	.0177078	56	.090149	150	.241470	420	.676116	740	
12	.0193176	58	.093368	155	.249519	430	.692214	750	1.20735
13	.020927	60	.096588	160	.257568	440	.708312	760	
14	.022537	62	.099807	165	.265617	450	.724410	780	
15	.024147	64	.103027	170	.273666	460	.740508	790	
16	.025757	66	.106247	175	.281715	470	.756606	800	1.28780
17	.027366	68	.109466	180	.289764	480	.772704	820	
18	.028876	70	.112686	185	.297813	490	.788802	840	
19	.030586	72	.115906	190	.305862	500	.804900	860	
20	.032196	74	.119125	195	.313911	510	.820998	880	
21	.0338058	76	.122345	200	.321960	520	.837096	900	1.44882
22	.0354156	78	.125564	210	.338058	530	.853194	920	
23	.037025	80	.128784	220	.354156	540	.869292	940	
24	.038635	82	.132004	230	.370254	550	.885690	960	
25	.040245	84	.135223	240	.386352	560	.901488	980	
26	.041855	86	.138443	250	.402450	570	.917586	1000	1.60980
27	.043464	88	.141662	260	.418548	580	.933684		
28	.045074	90	.144882	270	.434646	590	.949782		
29	.046684	92	.148102	280	.450744	600	.965880		
30	.048294	94	.151321	290	.466842	610	.981978		
32	.051514	96	.154508	300	.482940	620	.998076		
34	.054733	98	.157760	310	.499038	630	1.014174		

TABLE LXXIV.

THEORETICAL KILOWATTS IN WATER FLOWING AT THE RATE OF ONE CUBIC FOOT PER MINUTE AND OPERATING UNDER VARIOUS HEADS.

Head	kw.	Head.	kw.	Head.	kw.	Head.	kw.	Head.	kw.
4	.00565	100	.1413	330	.4663	560	.7913	790	1.116
5	.00706	105	.1483	335	.4734	565	.7984	795	1.123
6	.00848	110	.1554	340	.4804	570	.8054	800	1.130
7	.00989	115	.1625	345	.4875	575	.8125	805	1.137
8	.01130	120	.1659	350	.4946	580	.8195	810	1.145
9	.01271	125	.1766	355	.5016	585	.8266	815	1.1516
10	.01413	130	.1837	360	.5087	590	.8337	820	1.159
11	.01554	135	.1907	365	.5157	595	.8407	825	1.166
12	.01695	140	.1978	370	.5288	600	.8478	830	1.173
13	.01837	145	.2049	375	.5299	605	.8548	835	1.181
14	.01978	150	.21195	380	.5369	610	.8619	840	1.188
15	.02159	155	.2190	385	.5440	615	.8690	845	1.195
16	.0226	160	.2260	390	.5510	620	.8760	850	1.201
17	.02402	165	.2331	395	.5580	625	.8830	855	1.208
18	.0254	170	.2402	400	.5650	630	.8902	860	1.215
19	.02684	175	.2477	405	.5720	635	.8970	865	1.2222
20	.02826	180	.2543	410	.5790	640	.9040	870	1.229
21	.02967	185	.2614	415	.5864	645	.9112	875	1.2364
22	.031086	190	.2684	420	.5935	650	.9184	880	1.2434
23	.03259	195	.2755	425	.6005	655	.9255	885	1.2505
24	.03391	200	.2826	430	.6076	660	.9326	890	1.2576
25	.0353	205	.2895	435	.61466	665	.9396	895	1.2646
26	.0367	210	.2966	440	.62173	670	.9467	900	1.2717
27	.03815	215	.3037	445	.6288	675	.9538	905	1.2787
28	.03956	220	.3107	450	.6359	680	.9608	910	1.2858
29	.04097	225	.3179	455	.6429	685	.9674	915	1.2929
30	.0424	230	.3245	460	.6500	690	.9750	920	1.2300
32	.0452	235	.3320	465	.6570	695	.9820	925	1.307
34	.04804	240	.3390	470	.6640	700	.9890	930	1.314
36	.05086	245	.3462	475	.6712	705	.9960	935	1.3211
38	.0537	250	.3530	480	.6782	710	1.003	940	1.3282
40	.0565	255	.3603	485	.6853	715	1.0103	945	1.3353
42	.05934	260	.3674	490	.6924	720	1.0173	950	1.3423
44	.06217	265	.3744	495	.6994	725	1.0244	955	1.3494
46	.065	270	.3815	500	.7065	730	1.0315	960	1.3565
48	.0677	275	.3886	505	.71360	735	1.1390	965	1.3635
50	.07065	280	.3956	510	.72060	740	1.0426	970	1.37
55	.0777	285	.4027	515	.7277	745	1.053	975	1.377
60	.08478	290	.4098	520	.7348	750	1.0600	980	1.384
65	.09184	295	.4168	525	.7418	755	1.067	985	1.391
70	.0989	300	.4239	530	.7489	760	1.074	990	1.399
75	.1059	305	.4310	535	.7560	765	1.081	995	1.406
80	.11304	310	.4380	540	.7630	770	1.088	1000	1.413
85	.1211	315	.4451	545	.7700	775	1.095		
90	.12717	320	.4522	550	.7770	780	1.103		
95	.1342	325	.4592	555	.7842	785	1.110		

is utilized in one hour we would have 396 horse-power. If this energy can be sold for lighting purpose and brings for example seven cents per horse-power-hour each acre-foot of reservoir area will bring \$13.86 each time it is passed through the turbines.

TABLE LXXV.

THE THEORETICAL KILOWATT-HOURS AND HORSE-POWER-HOURS PER ACRE-FOOT OF STORAGE AREA FOR DIFFERENT HEADS.

Head ft.	Energy. h.p.-hr.	Energy. kw.-hr.	Head ft.	Energy. h.p.-hr.	Energy. kw.-hr.	Head ft.	Energy. h.p.-hr.	Energy. kw.-hr.
4	5.5	4.103	150	206.25	153.863	580	797.50	594.93
5	6.88	5.129	160	220.	164.120	590	811.25	605.19
6	8.25	6.155	170	233.75	174.378	600	825.00	615.45
7	9.62	7.180	180	247.50	184.635	610	838.75	625.71
8	11.00	8.206	190	261.25	194.893	620	852.50	635.96
9	12.37	9.232	200	275.00	205.150	630	866.25	646.22
10	13.75	10.258	210	288.75	215.408	640	880.00	656.48
11	15.12	11.283	220	302.50	225.666	650	893.75	666.74
12	16.50	12.309	230	316.25	235.923	660	907.50	676.99
13	17.90	13.335	240	330.00	246.180	670	921.25	687.25
14	19.24	14.361	250	343.75	256.438	680	935.00	697.51
15	20.62	15.386	260	357.50	266.69	690	948.75	707.77
16	22.00	16.412	270	371.25	276.95	700	962.50	717.92
17	23.37	17.438	280	385.00	287.21	710	976.25	728.18
18	24.75	18.464	290	398.75	297.47	720	990.00	738.44
19	26.13	19.489	300	412.50	307.72	730	1003.75	748.70
20	27.50	20.515	310	426.25	317.98	740	1017.50	758.95
21	28.87	21.54	320	440.00	328.24	750	1031.25	769.21
22	30.25	22.566	330	453.75	338.50	760	1045.00	779.47
23	31.62	23.592	340	467.50	348.75	770	1058.75	789.72
24	33.00	24.618	350	481.25	359.01	780	1072.50	799.98
25	34.37	25.644	360	495.00	369.27	790	1086.25	810.24
26	35.75	26.670	370	508.75	379.53	800	1100.00	820.50
27	37.12	27.699	380	522.50	389.78	810	1113.75	830.76
28	38.49	28.72	390	536.25	400.04	820	1127.50	841.01
30	41.25	30.772	400	550.00	410.30	830	1141.25	851.27
32	44.00	32.824	410	563.75	420.56	840	1155.00	861.53
35	48.12	35.901	420	577.50	430.81	850	1168.75	871.79
40	55.00	41.030	430	591.25	441.07	860	1182.50	882.04
45	61.87	46.159	440	605.00	451.33	870	1196.25	892.30
50	68.75	51.288	450	618.75	461.59	880	1210.00	902.56
55	75.62	56.416	460	632.50	471.84	890	1223.75	912.82
60	82.50	61.538	470	646.25	482.10	900	1237.50	923.07
65	89.37	66.667	480	660.00	492.36	910	1251.25	933.43
70	96.25	71.803	490	673.75	502.62	920	1265.00	943.69
75	103.12	76.931	500	687.50	512.87	930	1278.75	953.95
80	110.00	82.060	510	701.25	523.13	940	1292.50	964.20
90	123.75	92.318	520	715.00	533.39	950	1306.25	974.46
100	137.50	102.575	530	728.75	543.65	960	1320.00	984.72
110	151.25	112.833	540	742.50	553.90	970	1333.75	994.98
120	165.00	123.092	550	756.25	564.16	980	1347.50	1005.23
130	178.80	133.348	560	770.00	574.42	990	1361.25	1015.49
140	192.50	143.605	570	783.75	584.68	1000	1375.00	1025.75

Comparison of the Value of Power when Expressed in Horse-Power per Year or Kilowatt per Year.

Since the power of a stream is usually sold in terms of kilowatt-hours, it is often necessary to transfer horse-power-hours to kilowatt-hours; for example, if one horse-power is used every hour of the year (8,760) there would be used 8,760 horse-power-hours per year. This expressed in kilowatt-hours would equal 6,535 kilowatt-hours. If one horse-power is used 10 hours a day for a year (3,598 hours) the total energy would be 3,598 horse-power-hours which is 2,684 kilowatt-hours.

Table LXXVI gives the value for different periods of use of one horse-power when sold at one cent per kilowatt-hour and vice versa.

For any other price, multiply the values given in Table LXXVI by the price in cents.

TABLE LXXVI.

Cost of Power for Different Periods of Use.

Hours one kw. is used per day.	Cost per kw. and h.p. per year at 1c. per kw.-hr.		Cost per h.p. and kw. per year at 1c. per h p.-hr.		Hours used per year.
	kw. (1)	h.p. (2)	h.p. (3)	kw. (4)	
24	$87.60	$65.70	$87.60	$109.50	8.760
10	31.30	23.47	31.30	34.12	3,130
8	29.20	21.09	29.20	36.50	2,920
8*	25.04	18.78	25.04	31.30	2,504
6*	21.90	16.42	21.90	27.37	2,190

* Including Sundays.

Application of Table LXXVI.

If horse-power is worth $50.00 for a year of 3,130 hours and, it is desired to know how much this will be per kilowatt-hour divide $50.00 by the price per horse-power in column 2. Thus,

$$\frac{\$50.00}{23.47} = 2.13 \text{ cents per kilowatt-hour.}$$

Again suppose the price to be $25.00 per kilowatt for a year of 2,920 hours the price per kilowatt-hour being desired, then $25.00 divided by the cost of one kilowatt at one cent per hour, that is, $29.20 gives 0.856 cents per kilowatt-hour.

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